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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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GENERAL CONTRACT SYSTEM VERSUS SEGREGATED CONTRACTS

By Ward P. Christie,* Assoc. M. Am. Soc. C. E.

To Be Presented March 7, 1928

Introduction

By laboring constantly for greater economy of design, the Civil Engineering Profession has developed an amazing fund of technical knowledge. This knowledge, applied, has made possible the structural growth of the country. Well-deserved commendation has come to the profession. However, it is possible that the very intensity of thought exerted along technical lines has paved the way for criticism, as well as for commendation.

The thought has been expressed more or less frequently, that the Civil Engineering Profession in general is too thoroughly engaged in its technical pursuits to school itself in other necessary learning. The inference is, that the engineer usually does not become acquainted with those elementary principles of business, which any man should know, whose life work is concerned with construction. It is not presumed, of course, that the professional man should array himself with a business education, but rather that he should have a better understanding of the fundamentals of business management and the competitive forces with which he deals.

It has been stated, whether properly so or not, that many economies gained through increased technical knowledge are lost by unsound practice followed in the actual administration of construction. Some grounds for such a statement seem to have been evidenced by the many conferences of construction. For example, certain customs used in awarding and administering contracts are evidently sound and economical, while others are inherently wasteful. It is now recognized that not only the amount of a bid, but even the actual cost of construction may be influenced greatly by general provisions and payment clauses of a contract. In fact, many factors affecting the ultimate cost of a project transcend entirely the field of design.

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in **August, 1928.**

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One of these factors, which is the subject of this paper, seems particularly important. It is the relationship that should exist between the various responsibilities of construction and the control of operations. In private corporate enterprise, this relationship is well established, but in the administrative work of construction, the issues are still debated. An effort is here made to throw some light on the subject by analyzing the two methods of contracting now in use. For want of a better name they are designated as the General and the Segregated Contract Systems.

Each of these two systems has its proponents and opponents among the business agencies of construction, and the engineer or architect should be on his guard to discern the motives of the individual or business group. Much has been written concerning them, and considerable has been enacted into codes of practice or law, but through it all apparently runs a forgetfulness of those who pay the bill.

The general contractor capable of financing and managing a project in its entirety prefers the general contract, because it creates a demand for his specific service. A sub-contractor may prefer the segregated contract because it enables him to deal directly with an owner, thus escaping certain abuses which can be practiced by an unscrupulous general contractor. Some architects or engineers may believe that their conceptions are carried out more satisfactorily by segregation, whereas others unquestionably lean toward it to escape the backfire of inadequate plans and specifications.

It is apparent that either the general contract or the segregated contract can be utilized to the specific convenience of one or another of the various agencies of construction, but as all those agencies are presumed to serve the owner, either public or private, and as the owner's money makes construction operations possible, it seems logical that his interests and not those of any individual, profession, or business group, should be the criterion for judgment. Using that conception as a premise, therefore, the following analysis of the basic features of the two systems is offered, with the hope that the conclusions drawn may at least stimulate collective thought.

Some of the points discussed pertain more directly to building work than to general engineering construction, but from them the engineer can readily draw proper analogies for his own field of operation. The subject is subdivided into the following sections:

Definition of a General Contract,
Definition of Segregated Contracts,
Function of a General Contractor,
Financial Responsibility,
Legal Considerations,
Delay in Completion,
Relative Cost to Owners,
Relative Quality, and,
Accident Prevention.

Definition of a General Contract

Building Construction.—The general contract in building operations is an agreement entered into by an owner and a single construction agency whereby

that agency agrees to deliver to the owner a structure fully completed in accordance with the plans and specifications of the designer. It places upon this single agency, known as a general contractor, the task of supplying by such agencies as he may select, all construction equipment, materials, labor, special services, and appliances necessary to complete the project; and, furthermore, obligates this agency to protect the owner against loss from suits, claims, liens, and other encumbrances.

The outstanding characteristic of the general contract in comparison with other types is centralized responsibility for financing, for general management, and for detailed timing of installations whereby the cost of the work and the time of completion may be controlled according to some rational and comprehensive plan.

Engineering Construction.—The general contract in engineering construction establishes duties and responsibilities similar to those in building work, but may cover either an entire project, or merely one of a number of similar sections into which the work is divided by reason of its magnitude. The significant feature of the general contract in engineering construction, as elsewhere, is that it places under a centralized management and co-ordinating control those operations which depend upon each other or interlock in time and place.

Obviously, some engineering projects have natural segregations of work entirely independent of each other, with respect to interlocking, which makes them suitable for separate contracts. Such projects are, of course, excluded from this analysis, as they do not conflict with the principles herein stated with respect to interlocking work.

Definition of Segregated Contracts

The term, segregated contracts, is applied to the system of contracts whereby an owner enters directly into agreement with a number of independent contractors for the construction of the component parts of a project. Under this system no contractual relation exists between the independent contractors and, generally speaking, each is an agent of the owner. The function of co-ordinating their operations, or, in other words, the general management of the project, is performed by an agent of the owner.

In comparatively few cases of building work is the complete segregation of contracts carried out, as it is generally recognized that there must be some kind of central management by an agency that is financially responsible. Certain parts are awarded to concerns that usually operate as sub-contractors, and the remaining major portions are awarded to some principal contractor. This partial segregation, however, is subject to the same reasoning as the complete process, because the difference is only a matter of degree.

Function of the General Contractor

When properly functioning, the general contractor finances the work during construction, performs certain parts of the work himself, and co-ordinates the operations of all agencies engaged on the project, so that it may be constructed properly as a whole. Also, he assumes certain risks, peculiar to a business based on selling futures, and certain others, also, which arise merely from the presence of a number of different contractors operating on the same

site. Assumption of these risks in reality constitutes an insurance business of the most hazardous kind, for which no commercial company would attempt the underwriting. As an owner cannot obtain commercial insurance against them, they are pooled and assumed in the service of the general contractor. Among the contingencies against which he protects the owner are: (*a*) Increased cost of construction; (*b*) delay in completion; (*c*) injury to workmen and the public; (*d*) patent infringement; (*e*) injury to adjacent property; (*f*) damage from the elements; (*g*) imperfect material and workmanship; (*h*) liens and the abuse of lien laws; (*i*) default on subordinate contracts; (*j*) unknown underground conditions; (*k*) demands for extortionate wages; (*l*) fire and theft; and (*m*) labor, material, and transportation shortage.

Contrary to popular conception, the principal function of the general contractor is not to erect steel, brick, or concrete, but to provide skillful centralized management for co-ordinating all the various trades, timing their installations, and synchronizing their work according to some predetermined plan. In other words, the essence of general contracting lies, not in the broking of sub-contracts or the furnishing of labor, materials, and equipment, but in the highly specialized function of management, the success of which depends on the personal skill and direction of capable executives.

Due to the great number of different trades entering into construction, particularly building work, all of which must be co-ordinated with the main skeleton and principal elements, those principal portions are usually performed by the general contractor's own forces. To carry out his commitment with certainty, he must have actual physical control of these parts. Mere legal or contractual control without physical facilities for performing almost any part of the work does not assure completion within the time or at the cost intended.

The mere broker of contracts cannot successfully perform the service of a genuine general contractor. If one branch of the work falls down, he cannot perform it himself, but must seek a new agency to perform it, which, at best, means serious delay. The same is true of engineers and architects, even if they have the experience and ability to manage a project. In some instances, engineers or architects have successfully performed this management function, although they have done so, not by reason of their technical skill and training, but in spite of it. They were good construction executives, as well as technicians, with an understanding of business management—a combination rarely found in either business or the professions. Ordinarily, the professional engineer's or architect's management of construction may reasonably be expected to succeed about as often as the contractor's execution of engineering or architectural design.

Financial Responsibility

The general contract holds one single agency, a general contractor, financially responsible for the completion of a project within a given time and at a specified amount. If either cost or time should over-run, it is the contractor's and not the owner's loss. This is not the case under segregated contracts wherein the designer performs the general management of the work. He does not presume to guarantee the cost or pay the penalty for delay, and,

of course, should not be expected to do so, as his services are supposed to be professional. When an engineer or architect awards interlocking portions of a project to several contractors and himself performs the managerial duties necessary to co-ordinate their work, he is not exercising professional duties, but the duties of a business executive.

Perhaps the designer himself has not always perceived or understood the contingencies and liabilities which a segregated job places on an owner; nevertheless they are there, economically, ethically, and legally, and the owner cannot escape them. If the designer assumed these liabilities, the case would be different. He would then be acting as a broker, whose profit is justified, not only by the service performed, but by the risk of business which he has assumed. In that event he might well repaint his shingle to signify that he is a "Contracting Engineer", or a "Construction Contract Broker", for he is no longer strictly speaking a professional man.

The liabilities here mentioned are not theoretical or imaginary, and, in many cases, have ultimately cost the owner enormous sums. Independent contractors operating each according to his own convenience cannot avoid interference with one another. The work of each depends to a great extent upon the performance of another, and friction between the different superintendents is almost inevitable. Issues arise over the use of hoists, the laying out of conduits, storage of materials, the installation of safety devices, and other matters. Such conditions result in confusion on the job, mutual interference among the independent contractors, injection of extra expense, and the filing of perfectly legitimate claims against each other or the owner. The owner generally cannot escape payment of such claims against him, because his own representatives are named as being responsible, and because there is no financially responsible agency between himself and the independent contractors to absorb the loss.

From the owner's viewpoint, however, the broker is a "safer bet" than the professional engineer or architect; for he guarantees the price, usually has business experience, and, in the last analysis, must pay the cost of his own mistakes. When the designer splits up the work among a group of independent contractors he is acting as a broker with all a broker's financial safeguards and guaranties removed. This aspect of segregated contracts, although apparently not recognized by some professional men who have favored them on occasion, is a most vital consideration with respect to the interest of a construction owner.

If the professional man desires to engage in business, he has of course a perfect right to do so, and doubtless should do so if his capabilities lie in in that direction; but to exercise the business function involved in segregated contracts, and to let an owner proceed in such cases without informing him of his liabilities, does not have the earmarks of fair play.

Legal Considerations

To a certain extent each contractor holding an independent contract with an owner is his agent. Consequently, if one interferes with another's opera-

tions or injures him, the owner may find that he must stand suit jointly with the offending contractor. Even if the offender's agreement provides that he shall protect the owner against loss from actions brought by other contractors, such an agreement may not actually insure the protection. Obligations and responsibilities devolving upon each of the several contractors on a specific project, are difficult to determine, and the offending contractor can usually manage to shift the responsibility for injury to the owner's own representative. Moreover, the true obligations and equities of the various agencies of construction are so confused in the minds of American Courts, that there is little hope of pinning liability where it belongs. This condition is evidenced in part by the steady trend of construction toward the settlement of disputes by arbitrators familiar with the industry.

In addition to damage suits, there is also the question of criminal liability in event of accident or disaster. Rarely, if ever, has a Criminal Court in the United States convicted a contractor, designer, or owner, whose criminal negligence or wilfully dishonest actions have resulted in death to third parties. Under segregated contracts, a lack of clearly defined responsibility between the designing and construction agencies, and failure to pin these responsibilities where they belong, have made it nearly impossible to convict a guilty party. The injurious effect of such a condition upon the integrity of the entire industry must be obvious.

If an owner is sued when segregated contracts are used, he is very likely to be doubtful whether he should stand suit with the offending contractor, or join with the plaintiff against him. A properly drawn general contract, however, centralizes all responsibility in a single agency whose obligations are relatively clear. That agency can be reached at law and held responsible by an owner should the performance be upset by ignorance, negligence, or dishonesty of any agency on the work. In other words, the mere splitting up of interlocking work among a group of independent contractors, under the supervision of the owner's own representative, may place him in an extremely unsafe legal position.

Delay in Completion

Completion of a construction project within the time set is nearly always an important consideration to the owner, and, in many cases, is a vital one. Owners are not particularly interested in collecting penalties, but in obtaining their projects when expected. If the various operations are not "time-schedules" in advance and constantly synchronized on the job, delay is inevitable.

All other factors being equal, it seems almost axiomatic that responsibility and control when vested in a qualified general contractor, whose own money is at stake, will insure timely completion more certainly than when vested in the owner's professional adviser. Without an existing organization trained for the work, and without equipment or other facilities for actually taking over some backward portion, the latter is practically helpless to control the time of completion.

The points here mentioned are generally recognized by engineers and architects, but there is another point that is quite commonly overlooked,

namely, delays which constantly menace construction operations in and around cities by reason of difficulties with labor. The complications of union and non-union labor, as well as open and closed shop conditions, may readily paralyze the separate contract job. No one cause is more fruitful of delay and increased cost than this, particularly when contractors from different localities are engaged on the same project. Instances are not uncommon of a project having been delayed for weeks, because one of the independent contractors on an open shop job is obliged to employ only union labor. This condition needs little comment.

Ramifications of labor problems are not confined to single companies, single communities, or single crafts. A strike on one job may be called for causes arising on some job in a distant part of the country. The labor problems of construction, like mechanical installations, require special skill and experience. They require proper contact between the employers in the various trades, functioning of their trade associations, conferences with labor representatives, sometimes an organized resistance, and, in fact, a continuous working together of all elements aside from their contact on the work. The handling of labor has become probably the most difficult phase of the construction business in many localities, and the professional man is not in a position to handle the situation.

Relative Cost to Owners

Various proponents of segregated contracts have commonly sought their adoption by citing specific cases wherein the total of separate proposals on a construction project was less than the proposal for a general contract. This, they claim, demonstrates the economy of the system, but, at the same time, they fail to state that there are far more cases wherein the general contract bid is lowest. Also, there is another angle to proposal prices, which overshadows initial comparison, as follows: Assuming for the sake of analysis that plans and specifications are correct and complete, then, under a general contract, the contract price is all that the owner is required to pay; but when several independent contractors are engaged on interlocking work under the direction of an architect or an engineer, the sum of their prices rarely, if ever, constitutes the total cost to the owner. The extra expenses, as already suggested, arise from delays, interference, misplaced work, and lack of co-operation between the independent contractors. Under a general contract this over-run, if present, is absorbed by the general contractor. Under segregated contracts, it can fall only upon the owner.

In addition to the concealed expense of segregated jobs, there is also to be considered their effect upon competition. In receiving proposals for a general contract the owner obtains a double competition: First, the competition of sub-contractors dealing with general contractors; and, second, competition between general contractors. Frequently, it occurs that the general contractor, considering a proposal for some part of the work too high, substitutes his own figures, and brings the cost of the work within them. It is commonly recognized that he usually receives lower bids from the various trades than an owner dealing directly. He has his regular contacts with sub-contractors

who wish to deal with him in the future, whereas the owners may never employ them again.

A great many of the most experienced general contractors do not bid on projects likely to be segregated, because they feel that efficient performance of their work in such event is practically impossible. The award of a single integral part to some non-co-operative individual, disrupts and increases the cost of their work. Thus, the owner loses valuable competition which would often bring in more satisfactory proposals.

Some members of the mechanical trades in seeking to establish segregated contracts have represented that general contractors add a margin to the sub-bids, for which no service is given, and that this margin—called profit—can be saved by dealing directly with sub-contractors. Some owners may not detect the fallacy of the statement, but the analytical mind of a professional man should expose it. Entirely aside from the management service of the general contractor, there are many other expenses on the work which this so-called profit must defray.

Inquiry among representative construction companies shows that the usual margin added to sub-contracts is about 5 per cent. This amount, although in some cases it may contain an element of profit, is in reality a service charge, to compensate for such items as superintendent, night watchman, hoisting, storage space, protection from weather, temporary heating, safety work, and other overhead expenses which are not included in the sub-contractor's bid. When it is recalled that the architect in private practice charges about 3 or 4% for handling sub-contracts direct, yet provides none of these things except superintendence, it is obvious that the 5% contains no great amount of profit.

Under partial segregation, the principal contractor knows that he will be required to furnish these items and, therefore, includes them in his estimate. Then, when the designer adds to this his 3 or 4% for supervising separate work, the owner pays almost a double charge for general supervision. All the independent contractors, if they have had previous experience with segregated contracts, are very likely to include some allowance for the contingencies of a poorly co-ordinated job. Income tax reports show that the average net profits in contracting have been less than 2%, and that approximately 30% of the companies made no profit in 1924; therefore, there is evidently some room for doubt about the saving of this profit under separate contracts.

The subject of cost under the two systems may be analyzed and debated endlessly, because opportunity is never afforded for a true comparison on identical projects under identical conditions. If, however, experience in other lines of business counts for anything, and if the common axiomatic principles of business management and economics are founded on fact, segregated contracts cannot avoid certain inherent inefficiencies and expenses which are eliminated by the general contract system.

Relative Quality

Workmanship and materials under the general contract system receive a double inspection; one by the engineer or architect to protect the owner and

another by the general contractor to protect himself—for he is responsible to the owner for the work of sub-contractors. The supervising agency evidently values this latter inspection, as it is required by a great majority of contracts.

Under separate contracts, the engineer or architect can doubtless secure a satisfactory quality in the work of a specific contractor, but quality in the specific trade does not insure quality in the project as a whole. Unless the various parts have been properly timed, joined, and finished as a whole, the value of the project is impaired.

A well-known architect has stated, with respect to buildings, that there never had been, and probably never would be, prepared a complete and perfect set of plans. Some adjustment, alteration, and modification in the field is necessary. Doubtless, the same is true of engineering plans. This condition seems to necessitate some central agency in the contract to produce a structure that is integrally sound.

Proper timing of operations is a necessity on which the quality of a building project particularly depends. Conduits must be placed, not when the plumber or the electrician feels like doing so, but when forms and structural parts are ready to receive them. The heating plant is essential to the drying of walls and the execution of certain other work. Plaster, marble, tile, and painting are also dependent on the heating plant and, if it is not installed at the proper time, the results are not satisfactory.

Under complete segregation, proper joining and finish, or cutting and patching as it is commonly called, is extremely difficult to produce. Each trade seems to consider itself at liberty to bore holes, but feels no responsibility for their filling and finish. Under partial segregation, where some principal contractor is held responsible for such work, this is also a problem, as such contractor lacks control over those who cut the structure. A number of construction companies that have been consulted declared that they would like to be rid of responsibility for the mechanical trades, were it not for the fact that, when those trades operate under independent contracts, efficient operation and satisfactory conduct of the work as a whole is impossible.

Accident Prevention

Owners may not know it, but they are the ones who pay the costs of accidents in construction. The cost of injuries and deaths finally finds its way back to them through high production costs, increased premium rates, and general inefficiency in construction. Safety work is, therefore, not only a humanitarian obligation, but an economic necessity. It can only be carried on successfully when centrally supervised and enforced throughout a project.

When several independent contractors are operating on the same site, intelligent, co-ordinated safety measures are neglected, and the rate of accidents increases. Responsible contractors have stated that proper safety measures are practically impossible without some centralized management on the job. whereby some one answerable for his mistakes can, if necessary, force the installation of safety measures.

In some States, failure to comply with existing safety regulations precludes protection under the Workmen's Compensation Act. The contractor and the owner in case of accident may be jointly sued for twice or triple the limitations of the Act. If the contractor sued is responsible and is acting in the capacity of general contractor, the owner is safe. No doubt exists concerning who is responsible for all parts of the work; but if there are a number of independent contractors, responsibility may not be located or placed, and the owner may be obliged to pay the damage.

Conclusion

As previously mentioned, individuals in one or another group participating in construction prefer one or the other system of contracting. The reasons of some are unselfish, whereas those of others are distinctly selfish. No group is entirely one way or the other, and, in each, there is a tendency to blame the deficiencies of human beings on a system of contracting.

When the sub-contractor seeks to establish segregated contracts because an unprincipled general contractor tries to drive his bid down to that of an irresponsible competitor, he should realize that dealing directly with an owner is not a remedy. Owners or their representatives commonly do the same to general contractors, or for trifling causes hold up large sums of his money. They will do likewise to the sub-contractor. A number of representative sub-contractors have stated that they prefer the general contract, when the owner employs a responsible general contractor.

The professional man who represents that an owner can profit by eliminating a general contractor on the grounds that he performs no necessary service, is at best uninformed. Construction service is composed of certain elements which represent relatively certain expenses. They may be re-arranged among various agencies, but they cannot be eliminated. The significant point to be considered is, which of these agencies can perform the various services most efficiently and most satisfactorily to an owner. No one presumes that a man trained to the contracting business can step into design and render proper service. Neither can the designer step into general contracting, without actual contracting experience, and perform proper management service for an owner.

The general contract system was not suddenly conceived and launched on the populace, but grew to meet the requirements of owners, through a long period of years. It has endured in the hands of individuals who were unorganized, when all industry around them had developed powerful trade associations. By its endurance has been demonstrated the soundness and economy of centralized construction management, and it should not be cast aside merely to suit the convenience of a specific trade or profession.

Rather than attempt to correct any group or individual annoyances by scrapping a system which is both theoretically sound and practical, the responsible elements of each group should co-operate in correcting abuses, particularly as those abuses will prevail under any system, until all elements concerned with construction wage war on ignorance and unethical practice within their ranks.

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PAPERS AND DISCUSSIONS

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THE O'SHAUGHNESSY DAM AND RESERVOIR*

By John H. Gregory,† C. B. Hoover,‡ and C. B. Cornell,§
Members, Am. Soc. C. E.

Synopsis

This paper describes the design and construction of a dam and reservoir built to increase the storage of raw water for the water supply of the City of Columbus, Ohio. The masonry portion of the dam is of the overflow gravity type, the spillway section, 879 ft. in length between end abutments, being spanned by a 12-arch reinforced concrete bridge. The crest of the spillway is 64 ft. above low water in the river and has a discharge capacity of 110 000 cu. ft. per sec. The reservoir, formed by the construction of the dam, is 8 miles in length, has a maximum width of 1 900 ft., and a total available storage capacity of 5 341 000 000 gal. The total cost of the dam and reservoir was $2 220 080.

Introduction

The water supply of the City of Columbus, Ohio, is taken from the Scioto River. In 1908 the city completed a five-year program of important water-works improvements which included the construction of a water purification and softening works and pumping station adjacent to the Scioto River, about 2 miles from the center of the city, and of a masonry dam and storage reservoir on the river about 4.5 miles above the purification works. These improvements were fully described in a previous paper presented to the Society.‖

In the fall of 1925 the city completed a second five-year program of important water-works improvements, this program including enlargement

Note.—Discussion on this paper will be closed in August, 1928.

* Presented at the Fall Meeting, Columbus, Ohio, October 12, 1927.
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‖ "The Improved Water and Sewage Works of Columbus, Ohio," by John H. Gregory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXVII (1910), p. 206.

of the water purification and softening works and the construction of a second masonry dam and storage reservoir on the Scioto River.*

It is the purpose of this paper to describe the construction of the second masonry dam and reservoir, known as the O'Shaughnessy Dam and Reservoir.

Names of Dams

To avoid confusion, in case reference should be made to the paper and the report referred to, it will be well to state how the names, by which the two masonry dams are now known, came to be given. In the report recommending the construction of the second masonry dam, it was referred to, for the purpose of description, as the Dublin Dam, the proposed location being about 2 miles north of the Village of Dublin.

When the second five-year program was begun in 1920 Mr. Jerry O'Shaughnessy was the Superintendent of the Columbus Water-Works. Mr. O'Shaughnessy had been in the employ of the Water-Works Department for nearly fifty years, rising from the humblest position to the highest, and it was one of his greatest ambitions that the second five-year program should be carried to completion while he was still Superintendent of the Department, the work having been organized under a separate bureau of the Water-Works Department. This was not to be. Mr. O'Shaughnessy died on January 28, 1921, more than a year before the second dam was put under contract. The City Officials wished to recognize Mr. O'Shaughnessy's long years of service to the City of Columbus and decided to name the new dam for him. A bronze tablet, setting forth the action of the City Council in naming the new dam for Mr. O'Shaughnessy, has been mounted on the dam and bears the following wording:

O'Shaughnessy Dam

By Resolution of the Council of the City of Columbus, Adopted, February 7, 1921, This Dam Was Named and Designated the "O'Shaughnessy Dam" in Honor of the Late Jerry O'Shaughnessy, Who so Faithfully and Well Served the City as Superintendent of Its Water Works for Many Years.

In the paper describing the construction of the works built under the first five-year program, beginning in 1904, the first masonry dam was referred to as the Scioto River Storage Dam and it was built while the late Julian Griggs, M. Am. Soc. C. E., was Chief Engineer of the Board of Public Service, the work under this program having been organized as practically a separate branch of the Engineering Department of the Board. It was largely through the untiring efforts of Mr. Griggs that the first five-year program of improvements was started, and the first masonry dam was completed while he was still Chief Engineer of the Board. Mr. Griggs died on December 4, 1922. Having honored Mr. O'Shaughnessy for his long service to the City by nam-

* This second five-year program was carried out substantially as recommended in a "Report on Enlargement of the Water Supply, Purification and Pumping Works of the City of Columbus, Ohio," by John H. Gregory, dated August 2, 1919.

ing the second dam for him, the City Officials then wished to honor Mr. Griggs for his very valuable services in connection with the construction of the first dam. Hence, the first dam was renamed the Julian Griggs Dam. The following is from the resolution adopted by the City Council on December 18, 1922:

"That the present storage dam in the Scioto River be and is hereby designated as 'The Julian Griggs Dam' in honor of Julian Griggs, who so faithfully and well served the City as Chief Engineer and who was one of the prime movers in the movement to construct said dam, and who had general supervision of its construction."

The two dams and reservoirs will be referred to, therefore, as the O'Shaughnessy Dam and O'Shaughnessy Reservoir and the Julian Griggs Dam and Julian Griggs Reservoir.

Capacity of Works

In the report previously referred to it was recommended, in addition to enlarging the water purification and softening works and pumping station, that the water supply works be enlarged to provide for an average water consumption of 55 000 000 gal. daily. This recommendation has been carried out and the O'Shaughnessy Reservoir, together with the Julian Griggs Reservoir, provides sufficient storage to meet this requirement. With an estimated water consumption of 110 gal. per capita daily the source of supply will now provide for a population of 500 000, estimated to be reached somewhere between 1950 and 1955.

In the operation of the purification and softening works and the pumping station, water is used for washing filters, flushing mixing chambers and settling basins, boilers, etc., and a study of the past operation of the Columbus works indicated that an allowance of 5%, or 2 750 000 gal. daily, should be made for water so used. Consequently, the water supply works will provide for an average raw-water draft of 57 750 000 gal. daily, corresponding to an average water consumption of 55 000 000 gal. daily.

In the carrying out of the second five-year program the water-purification and softening works were enlarged, and, although it is not the intention in this paper to describe the enlargement of these works, their capacity before and after enlargement will be given, as a matter of reference. As first built, the purification works included ten rapid sand filters, each with a then rated capacity of 3 000 000 gal. daily, or a total of 30 000 000 gal. per day. While the piping and the filter rate controllers will permit running the filters at a somewhat higher output for a short period, in case of emergency, experience gained in the operation of the works for the first ten years demonstrated that it is not safe to run the filters, under Columbus conditions, at a rate greater than 30 000 000 gal. daily for any considerable time. The maximum capacity of the first filter installation, therefore, must be put at 30 000 000 gal. daily.

The average daily capacity of a water purification works to supply filtered water is dependent on the maximum daily filter capacity, that is, the maximum safe rate at which it is possible to filter water, on the volume of filtered

water stored in the filtered water reservoir and available for equalizing the fluctuating rates of water consumption, and on maximum and long continued high rates of water consumption. No general rule can be stated which will apply to all purification works, as the conditions at no two works are alike. From a study of Columbus conditions it was found that, with a maximum filter capacity of 30 000 000 gal. per day, an average daily capacity of only 22 300 000 gal. could be relied on.*

In the second five-year program the purification works were enlarged to a maximum filter capacity of 54 000 000 gal. daily by the addition of eight 3 000 000-gal. daily filters. The existing 10 000 000-gal. filtered water reservoirs were enlarged so as to have a total filtered water capacity of about 16 500 000 gal. Under these new conditions, following the method referred to, the purification works now have an average capacity of 40 000 000 gal. daily. A third and final enlargement of the purification works to an average daily capacity of 55 000 000 gal. will bring them up to the capacity of the water supply works.

In 1926 the City of Columbus had an estimated population of about 285 000. The water consumption for the same year was 27 400 000 gal. daily, including water supplied to a population of about 15 200 outside the city limits.

Water-Shed and Flow of the Scioto River

At the Julian Griggs Dam the Scioto River has a water-shed of 1 052† sq. miles, and at the O'Shaughnessy Dam, 988 sq. miles. On the water-shed the rock is of a limestone formation and is exposed at many places. Where the rock is covered, the drift is of varying thickness. The water is hard and is unsatisfactory for domestic and industrial uses unless softened. The average total hardness of the water for the last 18 years, 1909-26 inclusive, was 272 parts per million, 160 parts being carbonate and the remaining 112 parts non-carbonate hardness. The maximum total hardness recorded on any one day was 495 parts per million. The average turbidity was 75 parts per million, but, during freshets and flood flows, turbidities in excess of 2 000 parts per million have been recorded. At such times, however, the total hardness drops to less than 100 parts per million. The maximum recorded turbidity was 2 250 parts per million. The water is softened and filtered before delivery into the distribution system.

The daily flow of the Scioto River at the Julian Griggs Dam varies between very wide limits, from a minimum of 20 to 30 cu. ft. per sec. (roughly, 13 000 000 to 20 000 000 gal. daily), during the dry months of the summer and early fall to upward of 1 000 cu. ft. per sec. during the winter and early

* The method by which the average daily capacity was arrived at is a very interesting one and was described in detail, with graphs, by Mr. Gregory, in his report of August 2, 1919, on the enlargement of the water supply, purification and pumping works of the City of Columbus. So far as the writers are aware, that was the first time this method was used. A description of the method may also be found in Chapter XIV, "Relation Between Filtered Water Storage and Filter Capacity," by John H. Gregory and Robert B. Morse, Members, Am. Soc. C. E., in the Manual of the American Water Works Association (1925), pp. 330–345.

† In *Transactions*, Am. Soc. C. E., Vol. LXVII (1910), p. 218, the area was given as 1 032 sq. miles. The new area of 1 052 sq. miles is from measurements made on the more recently issued topographical maps of the U. S. Geological Survey.

spring months. Storage, therefore, is necessary in order to meet a water consumption demand of 55 000 000 gal. daily.

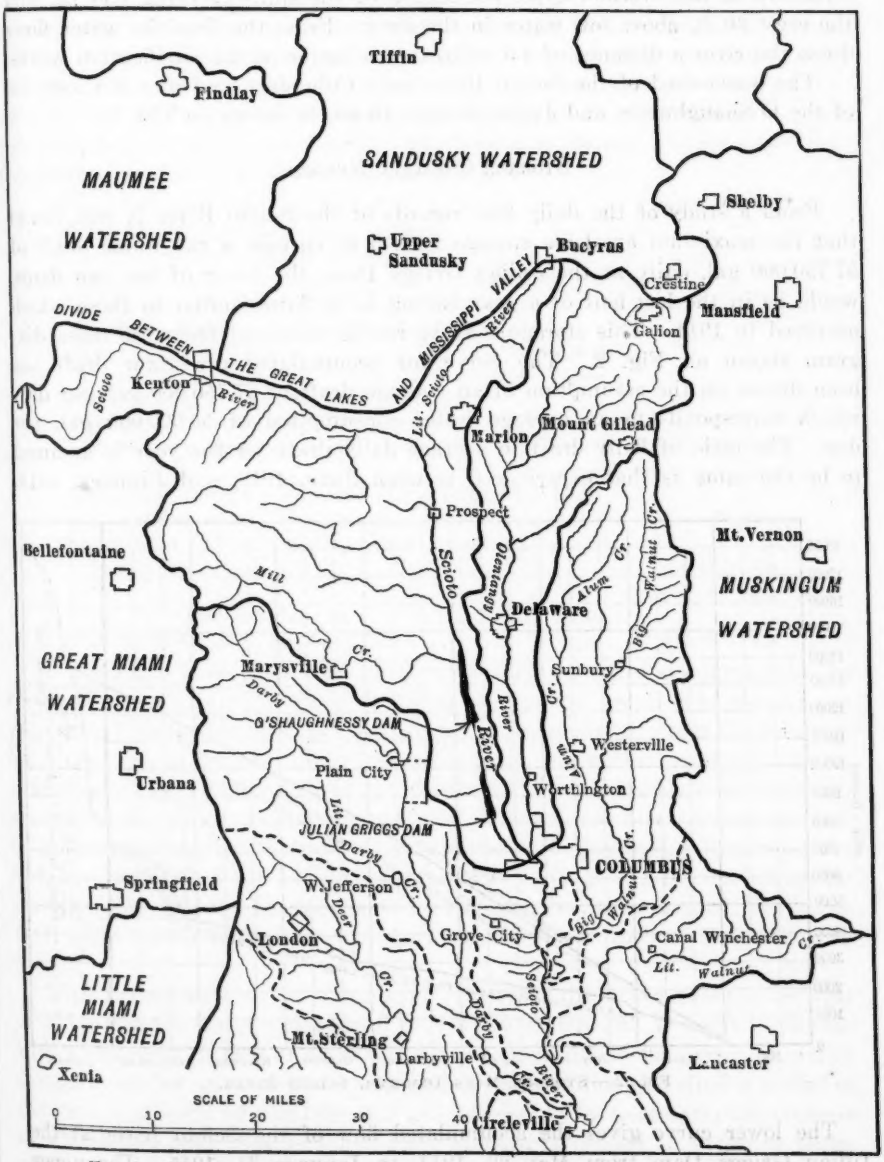

FIG. 1.—MAP SHOWING WATER-SHED OF SCIOTO RIVER, NEAR COLUMBUS, OHIO.

Beginning in February, 1909, records have been kept daily of the water level in the Julian Griggs Reservoir and of the opening of the sluice-gates in the dam when water was drawn from storage. From these records the

daily flow was computed. Although there are a few gaps in the record, they are only of minor importance. The Julian Griggs Dam is a curved masonry structure of the overflow type, the length of the spillway being 500 ft., and the crest 30 ft. above low water in the river. From the dam the water flows down the river a distance of 4.5 miles to the intake of the purification works.

The water-shed of the Scioto River near Columbus, and also the location of the O'Shaughnessy and Julian Griggs Dams, is shown on Fig. 1.

Storage Capacity Needed

From a study of the daily flow records of the Scioto River it was found that the maximum available storage needed to sustain a raw-water draft of 57 750 000 gal. daily at the Julian Griggs Dam, the lower of the two dams, would be in the last half of a year having daily flows similar to those which occurred in 1914. This storage may be readily obtained from the mass diagram shown on Fig. 2. The curve for accumulated raw-water draft has been drawn on the assumption of an average draft of 57 750 000 gal. per day, which corresponds to an average water consumption of 55 000 000 gal. per day. The ratio of daily draft to average daily draft for the year is assumed to be the same as that experienced between June, 1913, and January, 1914.

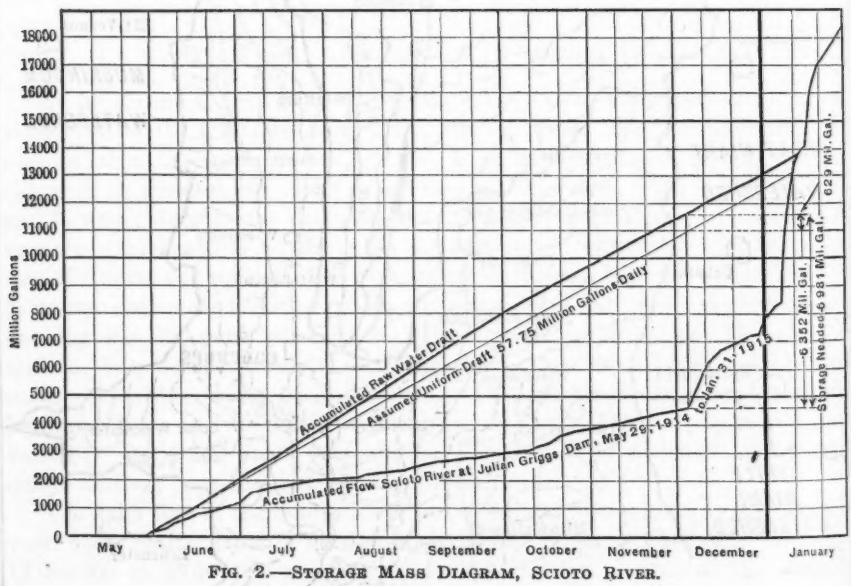

FIG. 2.—STORAGE MASS DIAGRAM, SCIOTO RIVER.

The lower curve gives the accumulated flow of the Scioto River at the Julian Griggs Dam from May 29, 1914, to January 31, 1915. The upper curve gives the accumulated raw-water draft for a similar time of the year, with an average raw-water draft for the year of 57 750 000 gal. daily, corresponding to an average water consumption for the year of 55 000 000 gal. daily. In computing the data for the plotting of this curve, it was assumed that

the ratio of the raw-water draft, day by day, to the average daily raw-water draft for the year would be the same as the ratios experienced on corresponding days in the period June, 1913, to January, 1914, it having been found, from a study of the daily water consumption records, that the maximum daily rates of long continued water consumption occurred in the period mentioned.

From these two curves it is seen that a total available storage of 6 981 000 000 gal. is needed above the Julian Griggs Dam to sustain a raw-water draft of 57 750 000 gal. daily. The available storage needed above the O'Shaughnessy Dam was then found as follows:

Total available storage needed, in gallons.....		6 981 000 000
Available storage at Julian Griggs Dam, without flash-boards, in gallons	1 487 000 000	
Additional storage at Julian Griggs Dam, with 4-ft. flash-boards, in gallons	500 000 000	
Total available storage at Julian Griggs Dam, with 4-ft. flash-boards, in gallons	1 987 000 000	1 987 000 000
Available storage needed at O'Shaughnessy Dam, in gallons		4 994 000 000
Storage at O'Shaughnessy Dam, assumed as not available, in gallons		250 000 000
Total storage needed at O'Shaughnessy Dam, in gallons		5 244 000 000

In plotting a mass diagram to determine the amount of storage needed to sustain a given rate of draft, it seems to be common practice to assume the rate of draft as the average for the year and, consequently, to plot the accumulated draft as a straight line. With such an assumption it is, of course, much easier and simpler to plot the diagram. That there may be, however, a considerable difference in storage needed, based on this assumption and on the assumption that the rate of draft varies from day to day in accordance with past experience, will be seen by reference to Fig. 2. To the mass diagram shown on this drawing has been added a straight-line, accumulated, raw-water draft based on the assumption of a uniform raw-water draft of 57 750 000 gal. daily.

With the assumption of a uniform raw-water draft throughout the year of 57 750 000 gal. daily a storage of only 6 352 000 000 gal. is needed, whereas with a variation in rate of draft day by day, throughout the period in which storage is needed, in accordance with past experience during a similar period of the year, a storage of 6 981 000 000 gal. is seen to be necessary, an increase of 629 000 000 gal.; that is, 9.9% more storage is needed in order to maintain higher rates of daily raw-water draft which are to be expected during the period when storage is necessary than under the assumption of a uniform daily rate of raw-water draft throughout the year.

From discussion with other engineers, as well as from examination of reports and books in which mass diagrams are given, it has appeared to

the writers that many have not appreciated that the assumption of a uniform rate of draft throughout the year may give too small a storage, about 10% in the case just illustrated. Therefore, they have taken this opportunity to bring the matter to the attention of engineers.

Type of Structure

The O'Shaughnessy Dam is a concrete masonry dam, of the gravity overflow type, straight in plan, and 1 750 ft. in length, including the approaches. The masonry portion of the dam is 879 ft. long, measured face to face of copings on the abutments. The two approaches have a total length of 871 ft., but they are not of equal length, the east approach being slightly longer than the west one.

Crossing the dam, above the overflow or spillway section, is a reinforced concrete arch bridge of twelve spans, each span measuring 64 ft. 6 in. in the clear. The net length of the spillway, under the bridge, after deducting the length of the bridge piers and the outlet gate-house, is 765.7 ft.

The crest of the spillway is 64 ft. above low water in the river on the down-stream side of the dam and 84 ft. above the rock foundation. The roadway on the bridge is 21.3 ft. above the crest of the spillway making, therefore, a total height of dam from rock foundation to roadway level of 105.3 ft.

A general plan of the dam is shown on Fig. 3, and a plan and elevation of the spillway and bridge on Fig. 4. A general view of the completed dam is given in Fig. 5, and Figs. 6 to 11, inclusive, are views of the dam at various stages of construction.

Capacity of Spillway

The most disastrous floods on record in the history of Central Ohio occurred in March, 1913. Columbus and Dayton were the two cities that suffered most. On the west side of Columbus great damage was done to property when the levees were overtopped and 93 lives were lost. The Olentangy River flows into the Scioto just above the center of the city and, at the confluence, the total water-shed is 1 612 sq. miles, 1 076 being on the Scioto and 536 on the Olentangy. Fortunately for Columbus the peak-flood flows of the two rivers did not occur at the same time, the Scioto following the Olentangy peak.

The peak of the Scioto flood occurred on March 25, 1913, with a flow estimated to have been approximately 80 000 cu. ft. per sec. at the Julian Griggs Dam, or about 76 cu. ft. per sec. per sq. mile of water-shed. At the peak, the depth of water passing over the 500-ft. spillway of this dam was 12.8 ft.

Assuming the same flood flow per square mile at the site of the O'Shaughnessy Dam, the discharge from the 988 sq. miles of water-shed above this point would have been about 75 000 cu. ft. per sec. After mature consideration of the subject it was finally decided to provide spillway capacity at the new dam for a flood of 110 000 cu. ft. per sec., or about 50% in excess of the 1913 flood, and also to increase the length of the spillway about 50% over that of the Julian Griggs Dam. Under these assumptions to discharge 75 000 cu. ft. per sec. over the spillway of the O'Shaughnessy Dam will

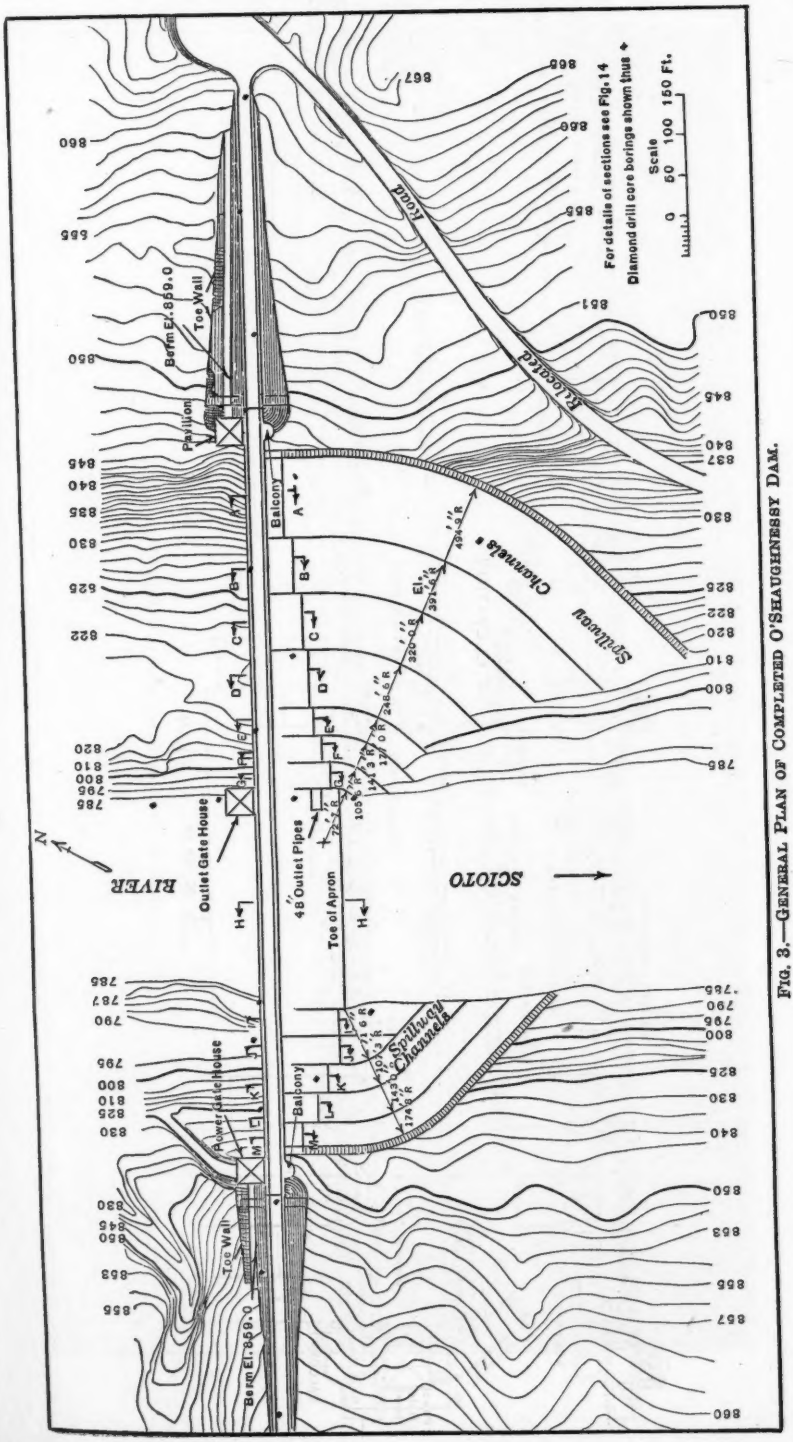

FIG. 3.—GENERAL PLAN OF COMPLETED O'SHAUGHNESSY DAM.

FIG. 4.—PLAN AND ELEVATION OF SPILLWAY AND BRIDGE, O'SHAUGHNESSY DAM.

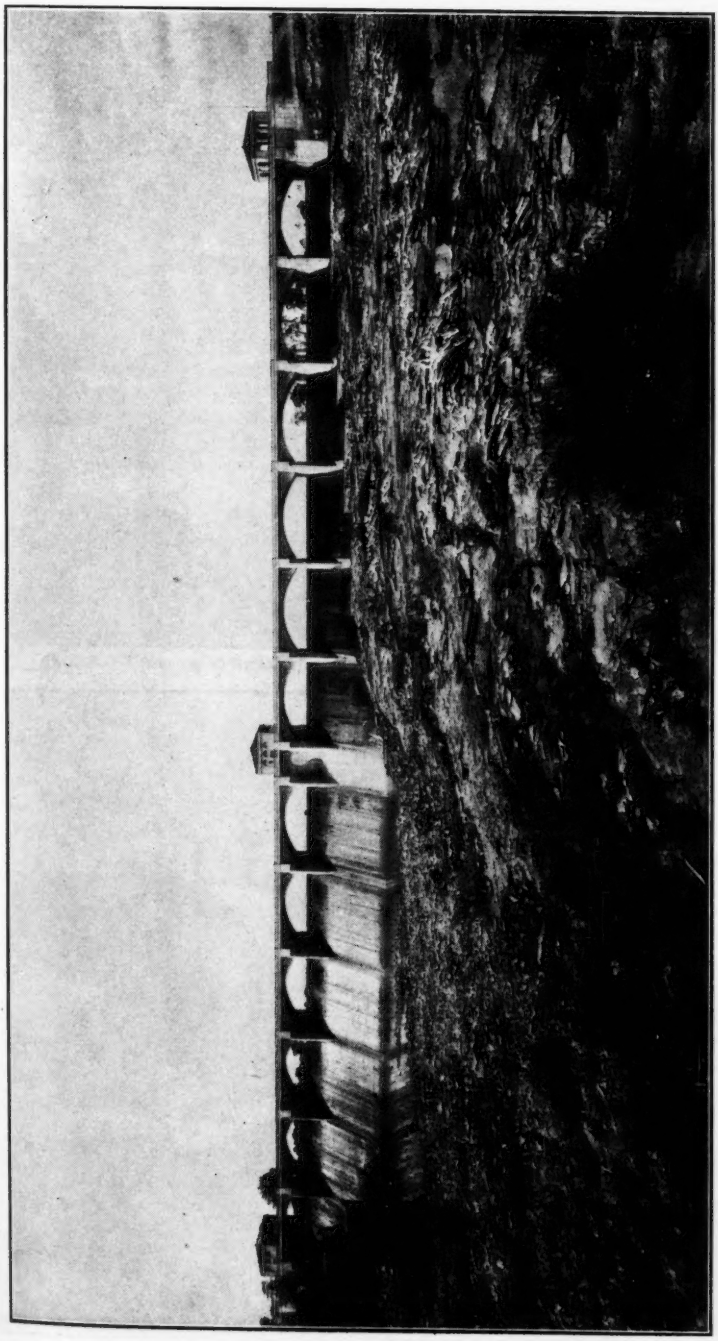

FIG. 5.—VIEW OF COMPLETED O'SHAUGHNESSY DAM.

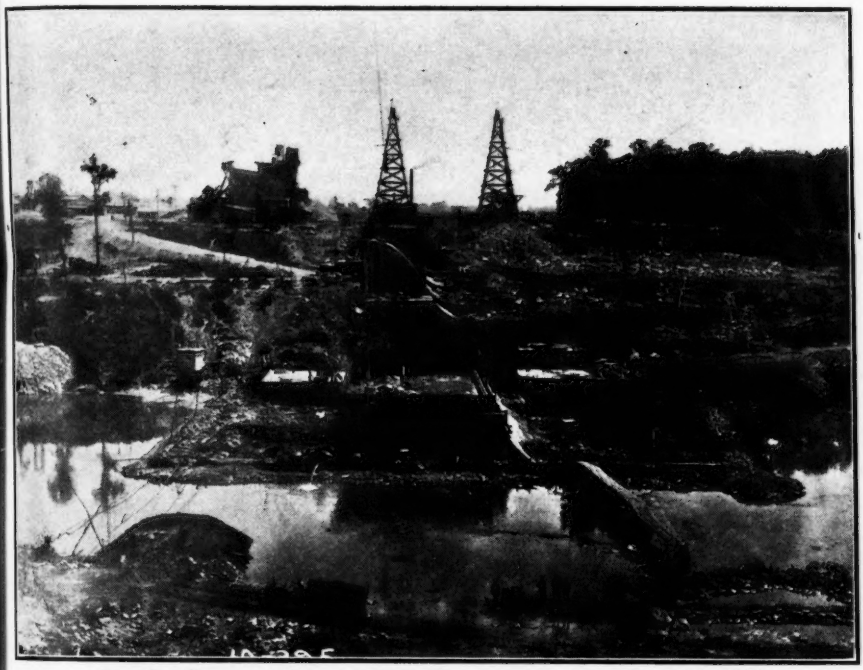

FIG. 6.—VIEW OF O'SHAUGHNESSY DAM, LOOKING EAST, JUNE 21, 1923.

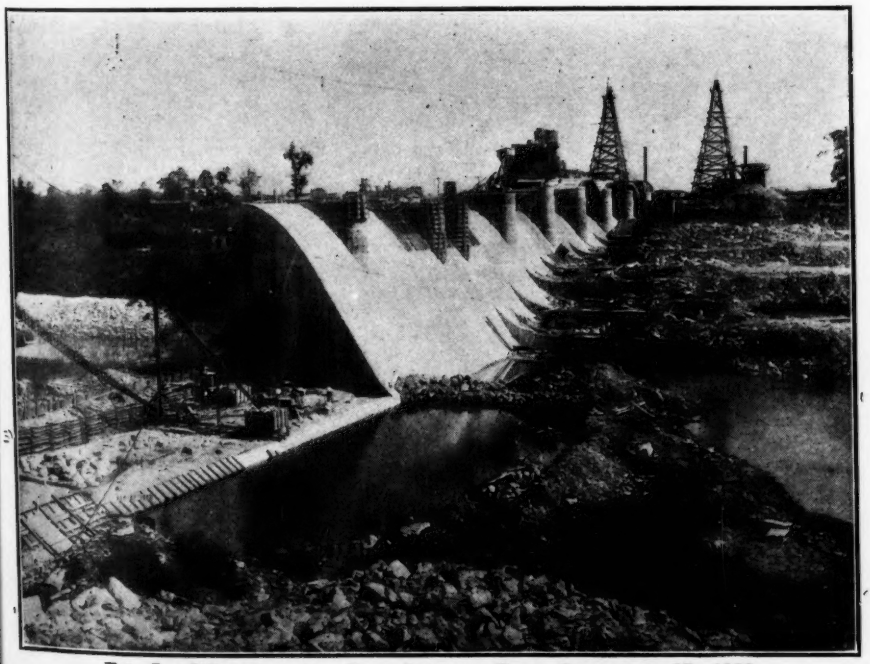

FIG. 7.—O'SHAUGHNESSY DAM, LOOKING EAST, SEPTEMBER 27, 1923.

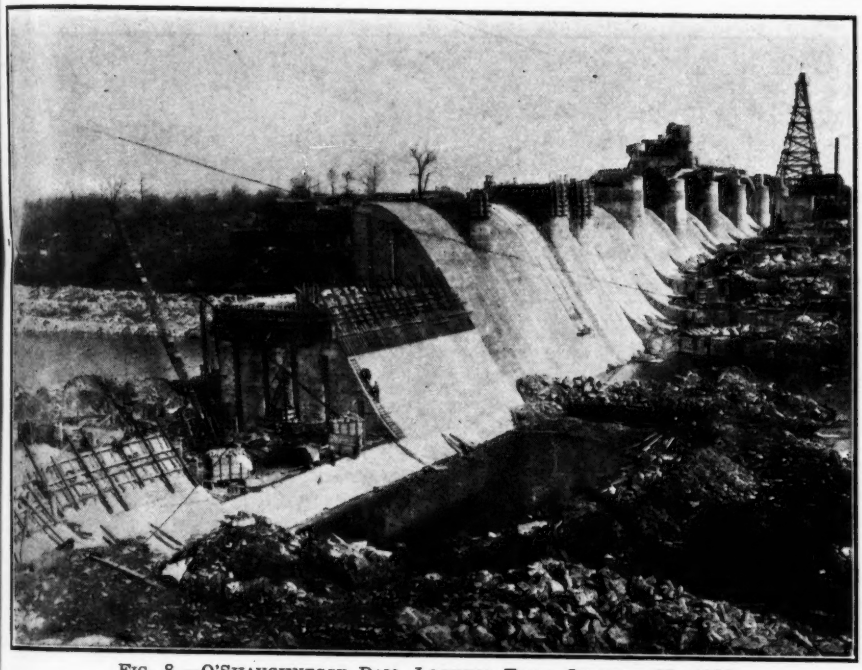

FIG. 8.—O'SHAUGHNESSY DAM, LOOKING EAST, OCTOBER 25, 1923.

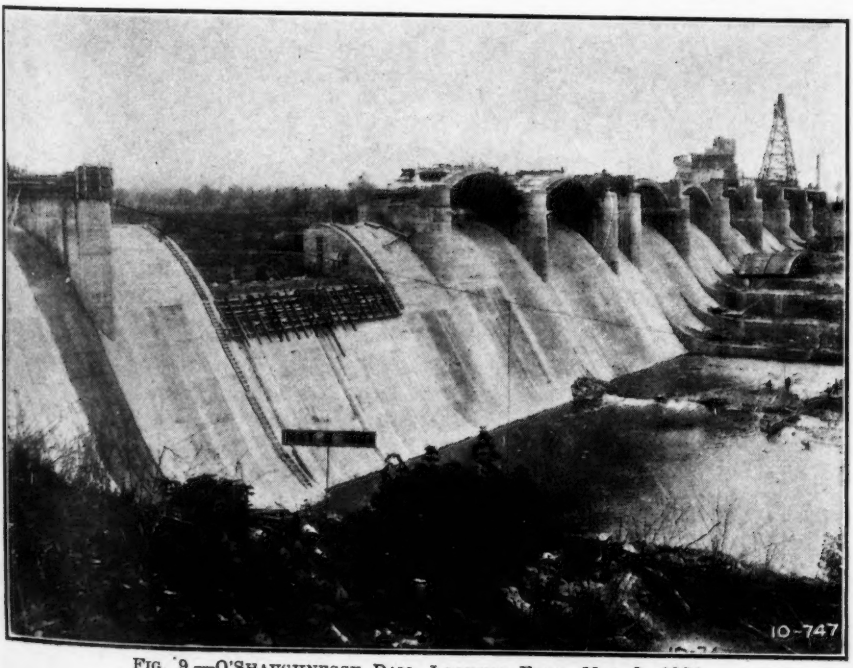

FIG. 9.—O'SHAUGHNESSY DAM, LOOKING EAST, MAY 9, 1924.

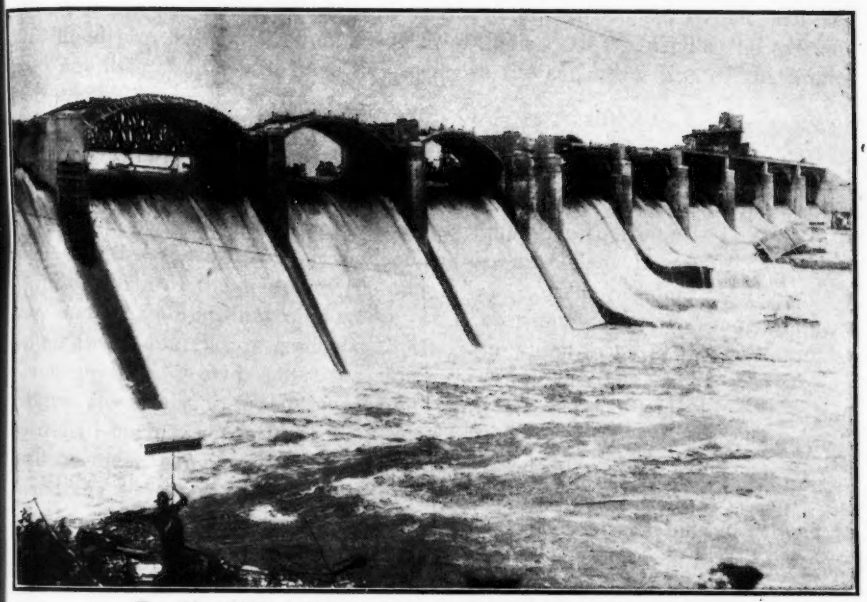

FIG. 10.—O'SHAUGHNESSY DAM, LOOKING EAST, JUNE 13, 1924.

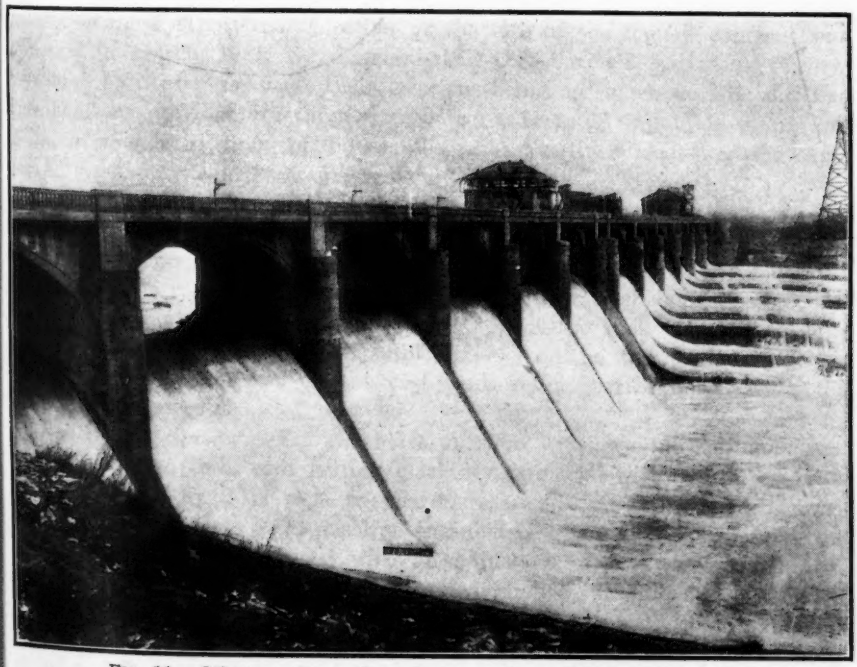

FIG. 11.—O'SHAUGHNESSY DAM, LOOKING EAST, FEBRUARY 13, 1925.

require a head of about 9.5 ft., and to discharge 110 000 cu. ft. per sec., the 1913 flood plus 50%, will require a head of about 12.5 ft. Under the extreme flood condition water will have to rise above the springing line of the arches of the bridge over the spillway.

Selection of Site

The site on which the dam is built was finally selected after more than a year of preliminary surveys, studies, and investigations. This site is about 4 miles above the upper end of the Julian Griggs Reservoir and about 16 miles north of the center of the city (Fig. 1). The problem resolved itself into one of finding that site where, at a minimum cost of construction, a suitable rock foundation could be obtained on which to build the dam and, at the same time, provide sufficient storage. Numerous sites above the Julian Griggs Reservoir were surveyed and more than 1 200 rod soundings and spud-drill borings made in order to locate the surface of the rock. The spud-drill borings, using an ordinary well-drilling machine, were made where rod soundings were not feasible. Comparative estimates of cost were then compiled and, on the basis of these, the site finally selected, was chosen, subject to revision, however, should diamond drill core borings reveal unsatisfactory sub-surface conditions.

Diamond Drill Core Borings

Following the tentative selection of the site of the dam a contract was awarded in March, 1921, for diamond drill core borings. All told, twenty-two such borings were made, their locations being as shown on Fig. 3. The specifications provided for three sizes of core, 1¼ to 1½ in., 1½ to 2 in., and 2 in. or greater in diameter, but as the price per linear foot was the same for all sizes, only 2-in. cores were taken.

Deep holes were drilled under the body of the dam, near the water's edge, on the east bank of the river; at the top of the two steep slopes on the east bank; at the top of the steep slope on the west bank; and at the center of the river. The remaining holes were of less depth, but all borings were carried to a somewhat lower depth than the top of the rock at the next hole toward the river, the holes at the top of the slope on the west bank, and at the top of the first steep rise on the east bank being carried down below the bottom of the river bed.

Drilling Equipment.—For the holes on shore two gasoline engine-driven, Sullivan friction feed, core-drilling machines, equipped with single core barrels and served by 2-in. pipe tripods, were used. Water for drilling was pumped from the river by a gasoline engine-driven, Gould triplex pump, the discharge pipe being equipped with an adjustable safety valve which permitted very even pressures to be maintained for both drilling and testing.

For drilling the hole in the river three lightly constructed high-sided dories were used. The gunwales of these boats were bored with pairs of holes at about 3-ft. intervals on both sides of the boats. Two of the boats were lashed together, pins placed in the spacer holes, light timbers placed between

the pins and lashed down, and 2-in. plank laid on the timbers to form a floating platform. The drilling machine was skidded on to this platform from the bank, the whole being towed to position by the third boat and anchored with four heavy ship's anchors.

The equipment furnished also included a small portable shop and tools for making repairs and for resetting the diamonds in the cutting bits, this work all being done on the job. The equipment for testing is described subsequently.

Drilling.—Where the rock was not exposed a 4-in. casing was driven to the top of ledge rock by one of three methods: First, by driving the casing, pulling it up, and knocking out the earth core, and then repeating the operation; second, by jetting out the earth and driving the casing; or, third, where slabs of rock embedded in the earth were encountered, by drilling with a spud bit. All casings were left in place and capped. In exposed rock, short pieces of casing were cemented into the holes in order to save them for grouting during construction.

The drilling of the holes was done in the usual manner, a complete log of each hole being kept, the notes reporting any changes in the color or quantity of the discharged water containing the cuttings, the depth to the bottom of the cutting bit at each pull, a sounded depth, and the length of core obtained at each pull, as well as notes on the physical characteristics of the rock, etc.

The cores were placed directly in boxes built for the purpose, marked blocks of wood being placed between the rock obtained from each pull; the boxes were covered with wire screening as soon as filled. The average depth of the holes was 34.55 ft., the maximum being 65.0 and the minimum 19.3 ft.

Hydraulic Tests.—Each hole was subjected to one or more hydraulic tests, water being applied under a pressure equivalent to a static head corresponding to Elevation 860, for the purpose of determining the water-tightness of the rock and also for locating any open seams or fissures. More and better information on these essential points was obtained from the hydraulic tests than from any other source, not excluding the information secured from the cores.

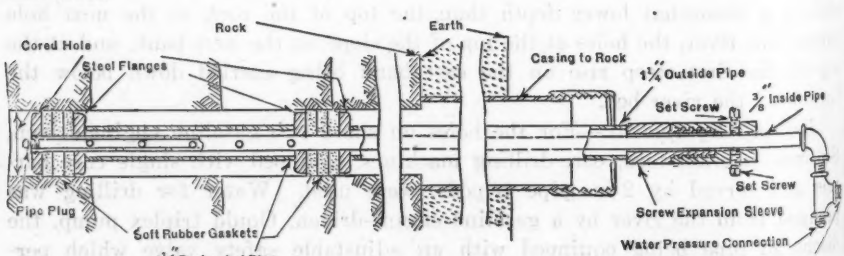

FIG. 12.—HYDRAULIC TEST APPARATUS FOR DIAMOND DRILL CORE BORINGS.

Water, under the pressure noted, was applied to the test hole through a double string of pipe of lengths necessary to reach the point to be tested. The outside pipe, shown in Fig. 12, was 1¼ in. in diameter and was made up

with a set of soft rubber gaskets at the bottom, a short length of pipe above, with several holes drilled through the sides of the pipe, a second set of gaskets, and a second piece of pipe reaching to above the ground. The gaskets were held between iron flanges, the gaskets and flanges having an outside diameter slightly smaller than that of the test hole. The inside pipe, $\frac{3}{4}$ in. in diameter and about 1 ft. longer than the outside string of pipe, passed through the upper set of flanges and gaskets, through the upper flange and gaskets of the lower set, and was screwed into and through the bottom flange. This inside pipe was closed at the lower end with a removable plug, was perforated with several holes just above the lower set of gaskets, and was fitted with an expanding screw sleeve at the top. The upper half of the sleeve could be securely fastened to the inside pipe with set screws, and the lower half bore directly against the top of the outside pipe. By screwing the sleeve out, the gaskets could be compressed until water-tight connections were made with the sides of the hole and with the outside of the inside pipe. Pressure could be applied from the inside pipe, through the holes in both pipes, to the rock sides of the test hole between the top and bottom gaskets, any desired length of hole being put under pressure by simply placing the desired length of perforated outside pipe between the top and bottom gaskets; or, the entire portion of the hole below the upper gasket could be put under pressure by removing the plug at the bottom of the inside pipe.

Water for testing was supplied under pressure through a short length of hose, the quantity and pressure being controlled by a gate-valve, and the quantity measured by a $\frac{3}{4}$-in. service meter. The pressure was shown on a small pressure gauge located close to the connection to the top of the $\frac{3}{4}$-in. inside pipe. Due to the small size of the inside pipe and the high friction loss, it was necessary to determine, in advance of testing the holes, the maximum quantity of water which various lengths of rig would discharge, taking into account the range of pressure required by the elevation of the ground at the several holes. Determinations, sufficient in number to cover the various lengths of test piping and pressures required, were made of the discharge capacity of the pipe by laying the pipe on the ground and measuring the discharge into the air, a constant pressure at the gauge being maintained by throttling the gate-valve.

Unless the existence of fissures or open seams in the rock had been determined during the boring of the holes, the first test was made at a depth of from 5 to 10 ft. below the top of the ledge rock, the entire hole below the upper gasket being put under pressure. If the hole was water-tight, or comparatively so, the pressure was maintained for 15 min., the leakage measured, and no further test of the hole made. If the leakage was equal in quantity to, or slightly less than, the discharge capacity of the test rig, additional tests were made at greater depths until the open seams or fissures were located. In a few instances, where there were indications that two or more small seams had been encountered, the pressure was confined to the space between the gaskets, with the gaskets set so as to be above and below one of the seams, the leakage into this seam measured, and then similar tests were made on the other seams.

Where seams were at a considerable depth, and it was desirable to determine the condition of the rock above the seam, a permanent seal was made by filling the hole with cement mortar to a point above such seam. After a sufficient time had elapsed to permit the mortar to set, the hole above the seal was tested in the usual manner.

Percentage of Core.—The length of core recovered, as measured in position in the core boxes, was 84% of the total depth of all the holes measured from the top of the solid rock to the bottom of the holes.

Cost.—The prices paid for making the diamond drill core borings, for temporary and permanent seals, for conducting hydraulic tests, and for casing left in place were, as follows:

Cores, per linear foot	$4.50
Temporary seals, each	15.00
Permanent seals, each	15.00
Hydraulic tests, each	15.00
Casing, left in place, per pound	0.06

In paying for the core borings the linear-foot measurement was made from the surface of the ground to the bottom of the hole. A temporary seal was paid for each time that the testing rig was set in a hole, the pressure turned on, and the flow measured. If the pressure was maintained and the flow measured for a period of 15 min. then payment for a hydraulic test was made, in addition to payment for the temporary seal. The total cost of the diamond drill core-boring investigation was $5 068, exclusive of engineering.

Geology of the Site

The diamond drill core borings and hydraulic tests made at the proposed site indicated that there were no sub-surface conditions seriously detrimental to its selection. The sub-surface investigation showed, however, that the grouting of the underlying rock would be required at certain points and that further extension of the grouted areas would be desirable, but as a measure of insurance rather than of necessity.

Before passing final decision on the selection of the site, and following the completion of the core borings and hydraulic tests, Professor J. A. Bownocker, of Ohio State University and State Geologist of Ohio, was requested to review the investigations which had been made, examine the cores secured, make such other investigations as might seem desirable, and report his findings. Professor Bownocker's report approved the site without qualification, but made the recommendation that the underlying rock be grouted for the purpose of preventing leakage through possible solution channels, cleavage planes, and open bedding planes in the rock.

From a point about 4.5 miles below the Julian Griggs Dam to a point well above the limits of back-water from the O'Shaughnessy Dam, the Scioto River flows through a valley cut in the Devonian limestone. The lower reaches of this course cut through the Columbus limestone at normal water level with the Delaware limestone outcropping in the higher lying portions of the

valley adjacent to the river. Near the Village of Bellepoint, about 6.5 miles above the site, the next lower formation appears in the bed of the river, namely, the Monroe limestone, the part here represented being of the Silurian Age.

Overlying the limestone, at all points in the valley where slopes are flat enough to prevent erosion, is a mantle of Wisconsin glacial drift, consisting very largely of yellow boulder clay with a thin top layer of clay loam, the boulder clay being an ideal material for embankments and the loam suitable for top dressing.

Near Columbus the Columbus limestone has an average thickness of about 105 ft. The Delaware limestone lies above the Columbus limestone, the contact being characterized by the bone-bed, notable among geologists for its high content of fossilized plates, teeth, and other bony structure of Devonian fish life. Below the Columbus limestone is the Monroe limestone, the two being separated, however, by an unconformity. The basal layer of the Columbus formation is commonly a conglomerate which consists of pebbles of Monroe limestone embedded in a matrix of smaller Columbus limestone and quartz sand.

The Columbus formation at the site consists of this basal layer, surmounted by many layers of limestone of varying thickness ranging from 2 or 3 in. to 4 and 5 ft. and varying in chemical composition from a true dolomite to an almost pure calcite, with physical characteristics almost as variable as the chemical composition.

Excavation for that part of the dam west of the outlet gate-house and in the easterly half of the river bed, developed the fact that the bed of the river, at the site, was in the massive lower layers of the Columbus formation and that excavation to the depth shown on the contract drawings would require the removal of much of this massive stone, a procedure which was unnecessary and which was immediately changed by decreasing the depth of excavation, where the rock foundation warranted, to a minimum of about 5 ft. under the dam proper, and with a somewhat less depth under the apron.

Excavation for the cut-off trench in the same location penetrated to a point about 1 ft. below the bottom of the basal conglomerate. The conglomerate was found to be from 6 to 8 in. in thickness and to have both above and below it, and throughout most of its extent, a thin layer of plastic blue clay. This discovery, which was not developed by the borings, led to the decision to carry all the cut-off trench excavation, across the river bed, to a point at least 1 ft. below the bottom of the basal conglomerate.

The dip of the rock in the Columbus District is from 20 to 30 ft. per mile to the east, or a little south of east. At the site it was found to approximate the larger figure and the cut-off trench was made to follow the variation in the level of the rock, being a little higher at the west end of the river section than at the easterly end under the outlet gate-house.

Excavation for the core wall at both ends of the dam encountered a small depth of the basal layers of the Delaware limestone underlaid by the

bone-bed and the top of the Columbus formation. The total thickness of the Columbus formation was about 87 ft., being less than the usual thickness for the Columbus District.

None of the excavated rock was too soft for use as coarse aggregate for concrete and the thicker layers made excellent stone for slope paving. It was necessary, however, to exclude the use of the softer stone for the wearing surface of the macadam roads constructed adjacent to the dam.

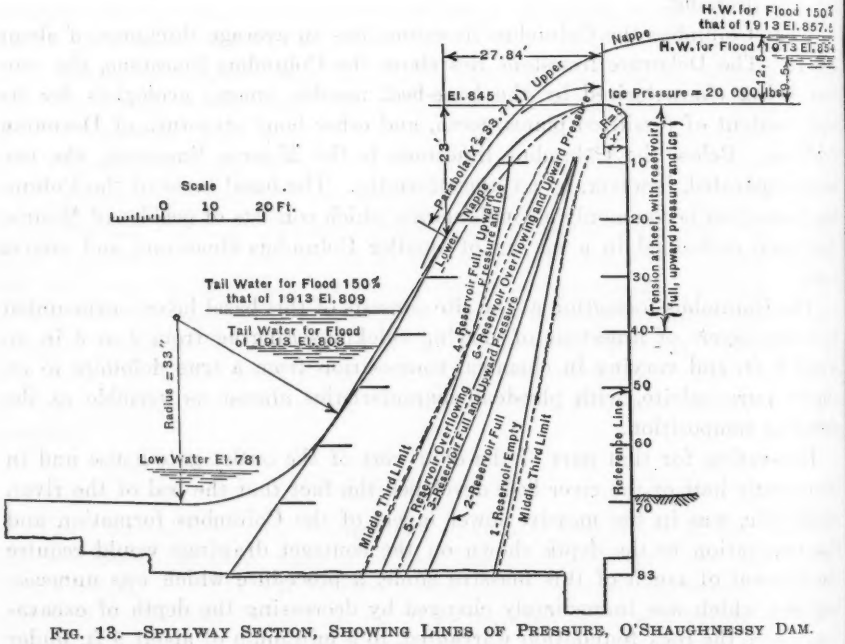

FIG. 13.—SPILLWAY SECTION, SHOWING LINES OF PRESSURE, O'SHAUGHNESSY DAM.

DESIGN OF SPILLWAY SECTION

The maximum section of the spillway, as constructed in the clear between the piers supporting the bridge above, is shown on Fig. 13. This section also shows the lines of pressure under various conditions of loading. In the design of the section the following assumptions were made:

Weight of masonry, in pounds per cubic foot..... 145
Weight of water, in pounds per cubic foot..... 62.5
Ice thrust, acting at crest, in pounds per linear foot.....20 000
Upward pressure: Full head pressure at heel decreasing uniformly to 0 at the toe, acting over two-thirds of the area.
Maximum head over spillway, 12.5 ft., for flood 150% that of 1913.
Surface of tail water, 36 ft. below crest, for flood 150% that of 1913.
No silt in reservoir.
No wind when reservoir is empty.

The lines of pressure shown are for six different conditions of loading as follows:

Case 1.—Reservoir empty.
Case 2.—Reservoir full.
Case 3.—Reservoir full and upward pressure.
Case 4.—Reservoir full, upward pressure, and ice.
Case 5.—Reservoir overflowing, 12.5 ft. above crest.
Case 6.—Reservoir overflowing, 12.5 ft. above crest, and upward pressure.

With the exception of Case 4—reservoir full, upward pressure, and ice—the lines of pressure lie, at all points, within the middle-third limits. In Case 4, the line of pressure is outside the middle-third limit down to an elevation 40 ft. below the crest of the spillway, which is at Elevation 845. Tension, therefore, would exist at the up-stream face of the dam above Elevation 805, but small in amount, not exceeding 26 lb. per sq. in. Steel reinforcement was provided, therefore, at the back of the dam at each horizontal construction joint above Elevation 805. The upper 10 ft. of the spillway was built monolithic between vertical expansion joints and was not reinforced.

The factor of safety against overturning, under the conditions of loading in Case 6, that is, reservoir overflowing, 12.5 ft. above crest, and upward pressure, is 2.25. The maximum pressure at the toe also occurs under Case 6 and is 5.82 tons per sq. ft. on a joint 83 ft. below the crest of the spillway. The maximum pressure at the heel, on this same joint, occurs under Case 1—reservoir empty—and is 6.92 tons per sq. ft.

In the lines of pressure shown, no allowance has been made for the weight of the overflowing sheet of water. Computations, however, were made to ascertain its effect, and it was found to add to the stability of the structure. At the bridge piers the lines of pressure would be slightly different due to the weight of the piers and bridge. Here, again, the added weight adds slightly to the stability of the structure.

On the spillway section, Fig. 13, are shown the upper and lower nappes for an overflowing sheet of water having a depth of 12.5 ft. corresponding to the 1913 flood plus 50%, and it will be noted that the shape of the crest is such that the face of the masonry projects into the sheet of water well beyond the lower nappe. The maximum and minimum sections of the spillway, as well as the intermediate ones, are shown on Fig. 14.

Reinforced Concrete Arch Bridge Over Spillway

At Powell Road, about 2 000 ft. above the dam, a county highway crossed the river on a 2-span steel truss bridge, 240 ft. in length. With the filling of the reservoir this bridge would have been submerged. As it was not feasible to abandon a crossing of the reservoir or river at or near this point, the choice then lay between raising the existing bridge about 51 ft., adding new trusses at each end, and building long expensive approaches, and the construction of a new bridge. Comparative estimates showed that it would be considerably cheaper to build a bridge across the dam and to abandon the crossing at

Powell Road. This solution of the problem was adopted after it had been submitted to and approved by the County Commissioners of Delaware County, in which the dam and reservoir are located. It was also of advantage to the city to have a bridge across the dam, as ready access would thereby be provided from one side of the dam to the other and also to the two gate-houses.

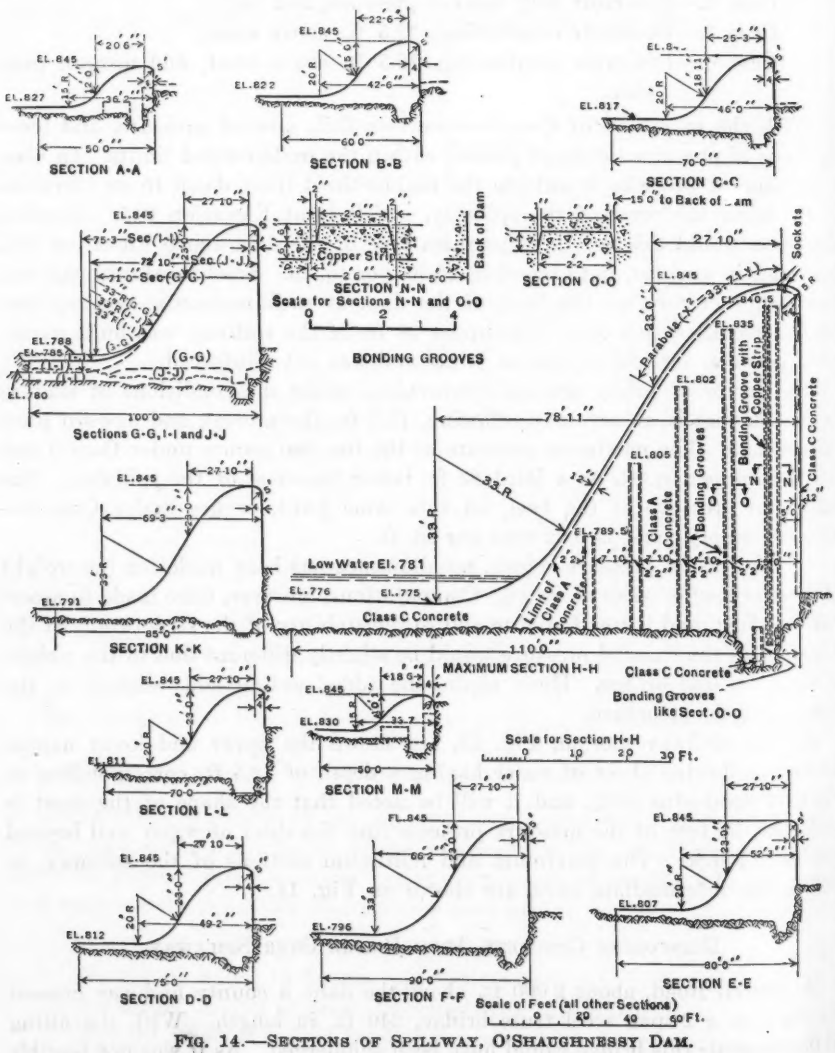

FIG. 14.—SECTIONS OF SPILLWAY, O'SHAUGHNESSY DAM.

The bridge consists of twelve reinforced concrete arches and has a total length of 879 ft., measured face to face of copings on the abutments at each end of the spillway. The arches are segmental, the intrados being struck on a radius of 52 ft. 9⅞ in. Each arch has a clear span of 64 ft. 6 in. and a rise of 11 ft. The thickness at the crown is 18 in., and at the springing line, 3 ft.

At right angles to the axis of the bridge, measured along the intrados, the arches have a length of 19 ft. 4 in.

The roadway over the arches has a clear width of 17 ft. 6 in., and is carried by reinforced concrete slabs supported by spandrel and curtain-walls extending down to the arch rings, except that, for a length of 21 ft. 6 in. at the center of each arch, the concrete slabs are supported on a cinder fill made directly over the arch rings. The roadway surface is a vitrified brick pavement, 4 in. thick, laid on a 1-in. sand cushion. The joints in the brick were grouted with an asphalt filler.

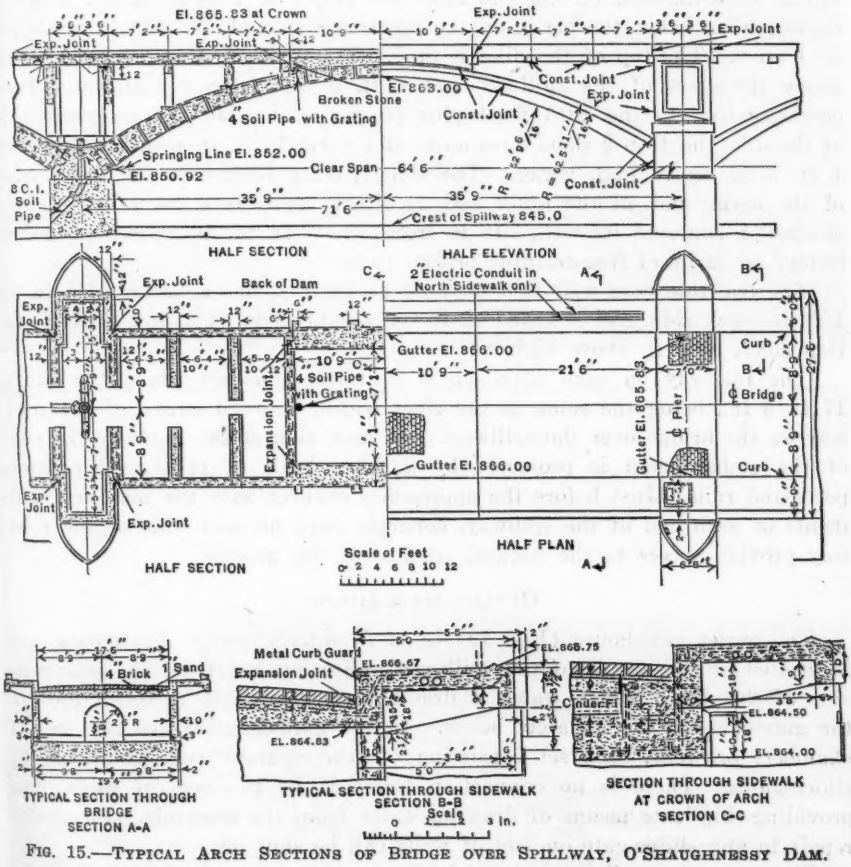

FIG. 15.—TYPICAL ARCH SECTIONS OF BRIDGE OVER SPILLWAY, O'SHAUGHNESSY DAM.

The roadway is drained by cast-iron inlets spaced at frequent intervals along each side of the pavement. The storm water is collected above the arches and is discharged through the arches, or through the piers, depending on the arrangement of the details.

On each side of the roadway is a 4-ft. sidewalk, consisting of a reinforced concrete slab carried on cantilever brackets anchored to the arch ring at the center of the arch and to the spandrel walls over the haunches

at other points. The balustrade on each sidewalk, together with the pylons, is constructed of blue Indiana limestone, sawed, planed, turned, or chisel-dressed, and finished by sand rubbing to a clean, smooth, uniform surface. The details of one of the arches is shown on Fig. 15, and a view looking along the paved roadway on the bridge in Fig. 16.

Approaches

At each end of the spillway section the approach consists of a rolled earth embankment or dike with concrete core wall, Fig. 18. The approaches are 25 ft. 6 in. wide on top and have side slopes of 1 on 2, except that the stone paving below the berm, on the up-stream side of the dike, has a slope of 1 on 3. The top of the dike is at Elevation 866.25, that is, 21 ft. 3 in. above the crest of the spillway and 8 ft. 9 in. above a high-water level corresponding to the 1913 flood plus 50 per cent. On the reservoir side of the dike the 1 on 2 slope terminates at a berm 10 ft. in width and located 1 ft. 6 in. above high water. The slope paving begins at the outer edge of the berm, and, at the lower end, is either keyed into the rock or abuts against a concrete toe-wall. It is from 12 to 18 in. in thickness and is bedded on 12 in. of free-draining broken stone.

The concrete core wall is 3 ft. wide at the top, has a batter of ½ in. in 1 ft. on each side, and is keyed 12 in. into solid rock. The top is at Elevation 858.5, or 1 ft. above high water.

The roadway on each approach is of bituminous macadam, the width, 17 ft. 6 in., being the same as the clear width, between curbs, of the roadway on the bridge over the spillway. On each side of the roadway the edge of the embankment is protected by a fence built of reinforced concrete posts and rails. Just before the approaches connect with the masonry abutments at each end of the spillway, concrete steps on each side of the roadway provide access to the natural surface of the ground.

Outlet Gate-House

The outlet gate-house (Fig. 19) is 32 ft. square, inside dimensions, and is located at the center of the spillway, projecting out from the up-stream side of the dam. When water is drawn from storage it is taken through the gate-chamber immediately below. In the gate-chamber are two sets of chambers or wells, each set consisting of two separate wells connected by sluice-gates. There is no connection between the two sets of wells, thus providing duplicate means of drawing water from the reservoir. In case of repair to any sluice-gate one set of wells can be shut off.

The up-stream well connects with the reservoir through six 4-ft. by 7 ft. 6-in. rectangular openings, and immediately behind these openings are three sets of stop-plank grooves. In the up-stream groove are placed coarse bar screens, Fig. 20, having clear openings of 3⅝ in. Water drawn from the reservoir, is discharged into the river immediately below the dam and flows into the Julian Griggs Reservoir. A fine screen, therefore, is not needed at present, and the coarse screen serves simply to keep out floating débris and thus protect the sluice-gates. The two inner grooves are for a double set of stop-

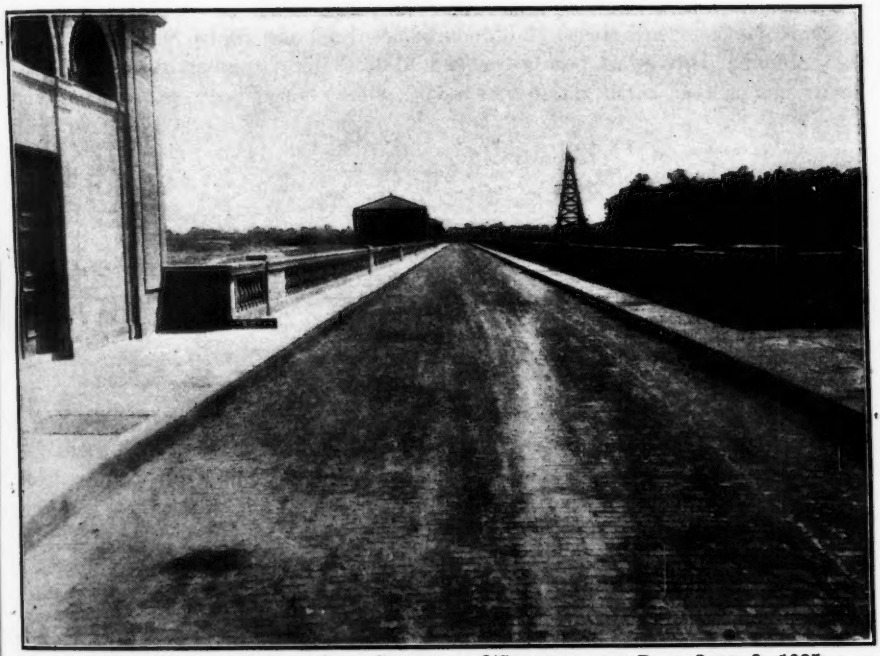

FIG. 16.—VIEW OF BRIDGE OVER SPILLWAY, O'SHAUGHNESSY DAM, JUNE 3, 1925.

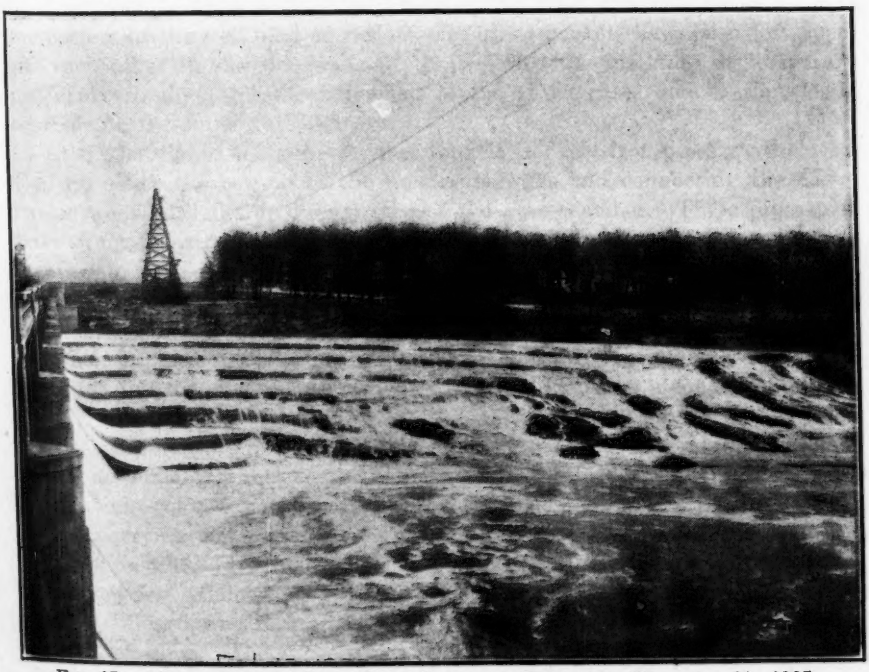

FIG. 17.—O'SHAUGHNESSY DAM, EAST SPILLWAY CHANNELS, FEBRUARY 13, 1925.

planks, Fig. 20, which can be inserted should it become necessary to inspect or repair a sluice-gate. The stop-planks are stored in grooves provided for the purpose in two special wells. The stop-plank lifter is also shown in Fig. 20.

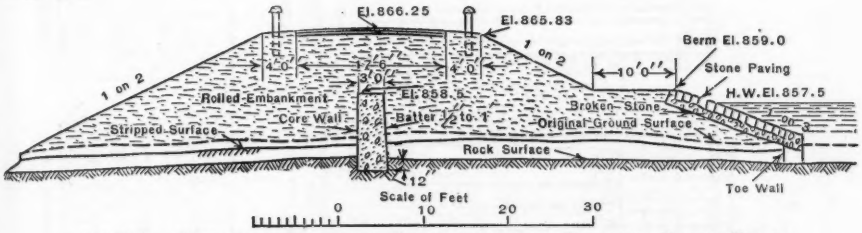

FIG. 18.—O'SHAUGHNESSY DAM, TYPICAL SECTION THROUGH EARTH DIKE.

Each up-stream well is connected with the corresponding down-stream well by three 36 by 60-in. rectangular openings, each opening being fitted with a rectangular sluice-gate of the same size as, and placed at the up-stream end of, the opening. These sluice-gates are set at different elevations so as to permit the drawing of water from storage at three different levels. From each down-stream well water is taken through two 30 by 60-in. sluice-gates into two 48-in., cast-iron pipes passing through the dam and terminating at the up-stream end of the apron. The ends are submerged so that the discharge is into the water cushion above the apron.

Each well is fitted with a ladder extending from the gate-house floor to the bottom of the well, and at various elevations pockets have been left opposite each other in the side walls of the well for the insertion of temporary platforms, so that access can be had to the sluice-gates and to the steady bearings on the sluice-gate stems.

At the bottom of the gate-chamber four 48-in., cast-iron pipes were placed, Fig. 19, below the bottom of the up-stream wells and connecting the down-stream wells with the up-stream face of the gate-chamber. These pipes took the ordinary flow of the river after the dam and gate-chamber had been constructed to an elevation above low water. After the completion of the dam each pipe was closed at the up-stream end by a cast-iron, blank flange, the pipe filled with concrete, and the lower end closed with another blank flange. Grout was then pumped into the pipe so as to fill it solid. In addition, the down-stream end of the pipe was permanently sealed, outside, by a concrete bulkhead grouted in.

The gate-house superstructure is built of cut and dressed blue Indiana limestone, matching that in the balustrade on the bridge, and is lined inside with a light-colored, salt-glazed face brick. The roof is constructed of steel trusses carrying concrete slabs 3 in. in thickness. The covering is of bright red mission tile laid on a bedding of Nalecode nailing concrete, 1½ in. thick. The doors and windows are of hollow metal construction built of heavy hard copper, stiffened, and reinforced. The windows are glazed with ¼-in. polished wire woven glass. The superstructure houses the hand-operated, ball-bearing, sluice-gate stands and also a 2-ton, hand-operated crane for raising and lowering the screens, stop-planks, etc.

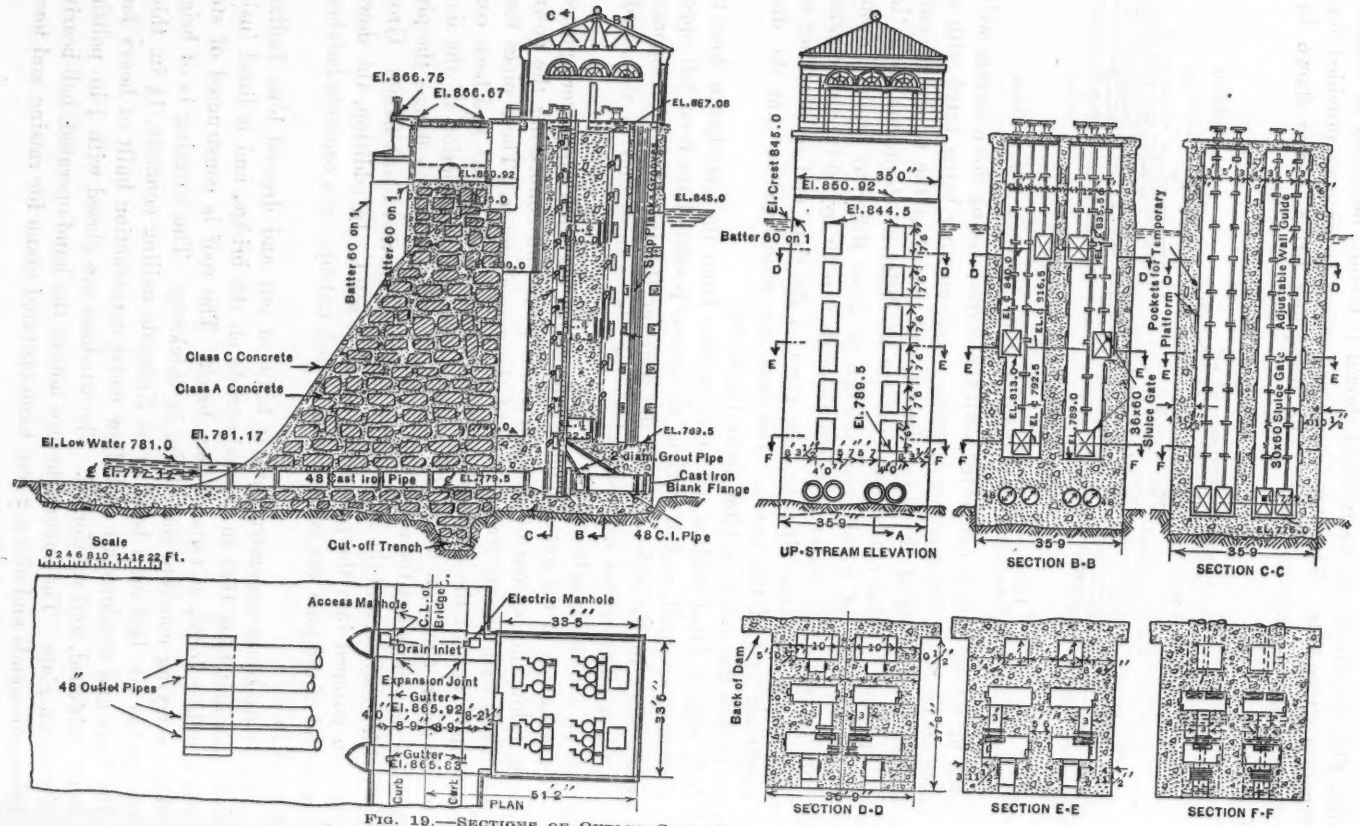

FIG. 19.—SECTIONS OF OUTLET GATE-HOUSE, O'SHAUGHNESSY DAM.

FIG. 19.—SECTIONS OF OUTLET GATE-HOUSE, O'SHAUGHNESSY DAM.

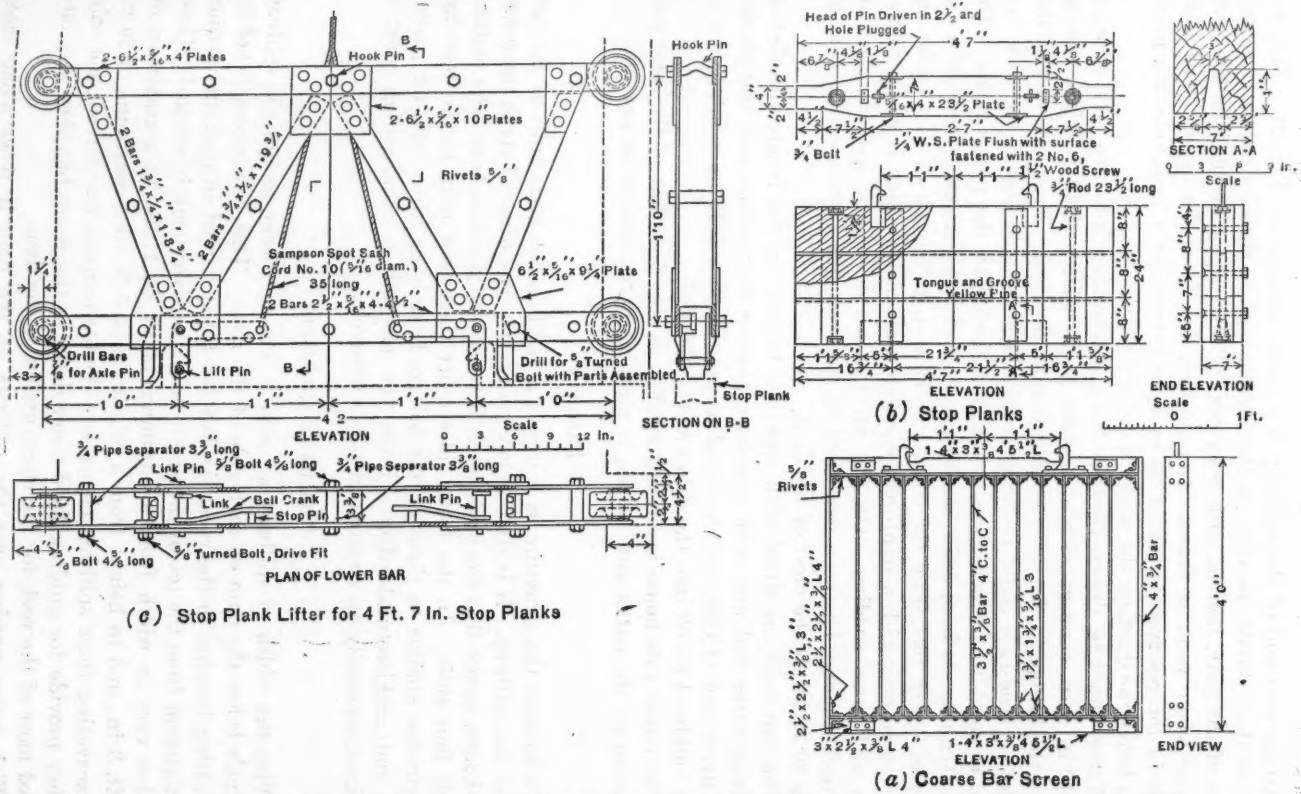

FIG. 20.—SECTIONS OF COARSE BAR SCREEN, STOP-PLANK, AND STOP-PLANK LIFTER FOR 4-FOOT 7-INCH STOP PLANKS.

Power Gate-House

At the west end of the spillway is a power gate-house, Fig. 21. This was built so that, should the city, in the future, wish to develop power at certain times of the year, that is, with reservoir full and water overflowing the spillway, or when water is drawn from storage, say, from the upper 10 or 15-ft. depth in the reservoir, it could do so by constructing a hydro-electric power plant below the dam, on the west side of the river.

Water would be drawn through the gate-chamber into two sets of wells, which, in turn, connect with a conduit, 15 ft. wide and 19 ft. high, passing under the bridge to the down-stream side of the dam. From the end of this conduit a head-race could be constructed along the side hill in the rock to a suitable point where penstocks could be extended to the power house. The head-race has not been built, but the upper end has been excavated for a distance of a few feet from the conduit. This short stretch of excavation has been roofed over with a reinforced concrete slab, above which is a covering of loam and grass sodding. On the up-stream side of the gate-chamber an approach channel has been excavated in the rock, the bottom of the channel being 20 ft. below the crest of the dam.

The two wells are fitted with cast-iron frames for the future installation of sluice-gates and also with cast-iron grooves for screens and stop-planks. The upper end of each well is closed at present with a 12-in. reinforced concrete bulkhead keyed into the side walls of the well.

The power gate-house is slightly smaller in size than, but of the same construction as, the outlet gate-house; it contains no equipment at present.

Pavilion

To balance the structure as a whole, a pavilion has been built at the east end of the spillway. It is similar in construction and dimensions to the power gate-house, except that the wall panels between the pilasters have been omitted on all four sides. In the three sides other than that next to the roadway, balustrades, similar to those on the bridge, have been built, both for protection and architectural effect. From the pavilion a commanding view is obtained, especially up stream across the reservoir.

Spillway Channels

With the width of spillway adopted it was necessary to provide spillway channels below the dam on each side of the river. This was accomplished by excavating benches in the rock, the benches being curved in plan and sloping down stream from the toe of the apron until the river bank is reached. These benches vary in width from a minimum of 32 ft. 11 in. to a maximum of 103 ft. 3 in., and in height from a minimum of 5 ft. to a maximum of 20 ft. In excavating these spillway channels a double purpose was served; not only do they provide for guiding flood-water down to the river, but they also furnished most of the rock for the construction of the dam.

With water passing over the spillway of the dam it flows along these benches and, at the same time, spills over from one bench to the next lower

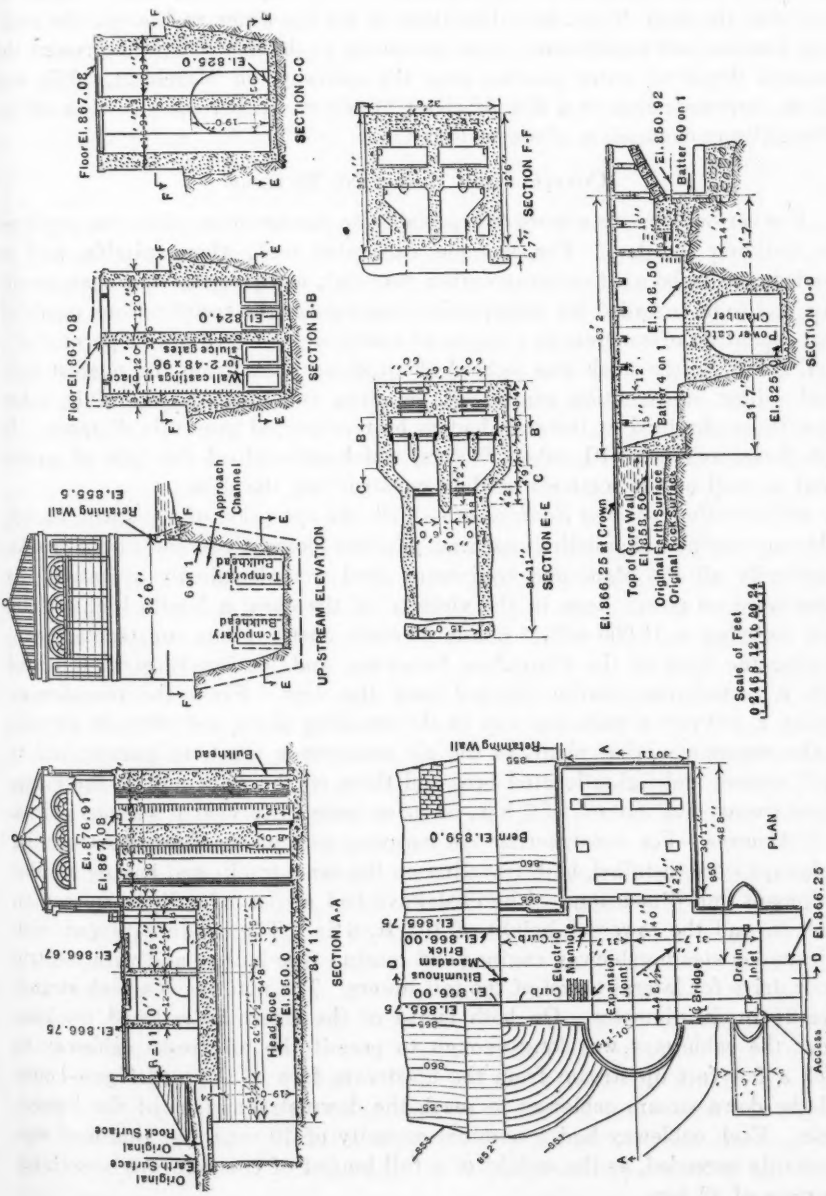

FIG. 21.—DETAILS OF POWER GATE-HOUSE, O'SHAUGHNESSY DAM.

one until it reaches the river. The greater the flow the longer the travel of the water longitudinally of the benches. With any considerable volume of flow over the dam, it is a beautiful sight to see the water rush along the spillway benches and tumble over from one bench to the next. To the present the greatest depth of water passing over the spillway, on March 21, 1927, was 4.2 ft., corresponding to a flow of about 21 500 cu. ft. per sec. A view of the east spillway channels is given in Fig. 17.

Construction Plant and Methods

For various reasons a more or less elaborate construction plant was required for building the dam. For example, excavated rock, when suitable, and as needed, was to be used as construction material; equipment, other than spouting, had to be installed for transporting concrete as the specifications provided that "spouting of concrete as a means of transportation will not be permitted"; and, again, as the work was located about three miles from the nearest railroad siding, construction equipment, supplies, and building materials, other than those obtained at the site, had to be transported over this distance. It was these, together with other, factors, which determined the type of equipment as well as the methods used in constructing the dam.

Electrically Operated Equipment.—With the exception of the steam shovel, cableway engines, channeling machine, gasoline locomotives, and motor trucks, practically all the plant and equipment used was electrically operated. As there were no power lines in the vicinity of the dam, a 7-mile high-tension line, carrying a 15 000-volt, 3-phase, 60-cycle current, was constructed, connecting the lines of the Columbus, Delaware, and Marion Electric Railroad with a transformer station located near the dam. From the transformer station a 440-volt circuit was run to the crushing plant and 220-volt circuits to the concrete mixing plant, to the air compressor plant, to pumps, and to small motors and lights located here and there on the work and in the camp. About twenty-five motors, of 5 h.p., or more, were in service at various points.

Cableways.—For constructing the masonry part of the dam two traveling cableways were installed, both traveling on the same tracks and having similar equipment and dimensions. The cableways had a span of a little more than 1 100 ft. and the towers a height of 103 ft. 6 in. They were equipped with Lidgerwood steam cableway engines and marine type boilers and with electric motor drive for lateral travel of the tail towers. The main cables, lock strand, were 2¼ in. in diameter. On both banks of the river the railroad trackage under the cableways was long enough to permit the up-stream cableway to reach a few feet up stream from the up-stream face of the outlet gate-house and the down-stream cableway to reach the down-stream face of the longest apron. Each cableway had a nominal capacity of 10 tons, but this load was frequently exceeded, as the weight of a full bucket of concrete was something in excess of 12 tons.

Transportation Equipment.—At the beginning of the work the transportation equipment consisted of a fleet of fifteen old White dump trucks; these were supplemented later by a varying number of locally owned trucks, two gasoline locomotives, trackage, and cars. On the east bank, immediately in

front of the cableway towers, two tracks were laid, one running to the concrete plant and the other to the crushing plant. Concrete was hauled, on one track, to either of the cableways in 5-yd. bottom dump buckets set on flat cars; the other track was used for carrying rock from the cableways to the crushing plant.

Trucks were used for hauling all the construction plant and equipment, as well as supplies and building materials, from the railroad spur at Powell, Ohio, 3 miles away; for hauling all the earth excavated, except a small portion used in the coffer-dams; and for hauling a considerable portion of the excavated rock either to the crushing plant direct, or to a point where it could be handled by the cableways. For the maintenance of the trucks a fairly well equipped garage, with repair parts and a few small machine tools, was built.

Excavation.—The bulk of the excavation, both earth and rock, was made with a 1-yd. Osgood steam shovel, mounted on a caterpillar truck. The excavated material was loaded into trucks or into skips handled by the cableways. The cableways delivered the excavated rock to cars on one of the tracks running in front of the cableway towers, or to a grizzly on the east bank where the larger stones were separated, for use in cyclopean masonry, before being discharged into the cars. The cars, in turn, delivered the excavated rock to the crushing plant.

The limestone encountered required extensive drilling and blasting. The drilling of holes for blasting and the drilling of all grout holes less than 15 ft. in depth, was done with jack-hammers driven by compressed air supplied from a stationary motor-driven compressor plant in which were located two, belt-driven, horizontal, Ingersoll-Rand air compressors. A Sullivan channeling machine was used to cut the sides of the cut-off trench, as required by the specifications. The mucking of the rock in the cut-off trench, and in the core-wall trenches, and the trimming of the excavation was all done by hand supplemented by light blasting. Excavation, both hand and machine, was carried on day and night during a large part of the 1922 and 1923 working seasons and during a part of the 1924 season.

Crushing and Screening Plant.—The principal items of equipment installed in the crushing and screening plant were a No. 6 and a No. 25 Kennedy gearless crusher, driven by 40 and 25-h.p. motors, respectively; a rotating cylindrical stone screen, 5 ft. in diameter and 19 ft. long, with a dust screen of larger diameter, 6 ft. long at the inlet end; a four-compartment storage bin and skip hoist; and bucket elevators and belt conveyors, all the equipment being motor-driven. Later, a small rotary screen was installed and used for washing the smallest size stone. The crushers were set at ground level and the screen at the top of the storage bin. The rock to be crushed was dumped into a receiving pit at ground level, hoisted by the skip to the charging platform over the crushers, put through the larger crusher, hoisted to the screen, and screened. The screened material went into bins from which it could be loaded into trucks or to storage piles, the smallest size aggregate being passed through the washer if desired. Tailings from the screen were conveyed to the smaller crusher, were recrushed, and passed through the screen again, together with the crushed stone from the larger crusher. The crushed stone

was screened into four sizes, screenings and dust less than ¼ in., ¼ to ¾ in., ¾ in. to 1½ in., and 1½ to 3 in.

Storage of Crushed Stone and Sand.—The storage plant for crushed stone and sand consisted of a timbered tunnel, with the top at ground level, extending from the concrete mixing plant to a point about 400 ft. distant from the plant; a timber trestle built directly over the first 225 ft. of the tunnel; a Green, portable, belt-conveyor unit; belt conveyors on the trestle and in the tunnel; and a bucket elevator extending from the end of the tunnel conveyor to the top of the concrete mixing plant. Crushed stone, more than ¼ in. in diameter, was delivered from the storage bins under the screen, by conveyor and elevator, to the trestle conveyor which, in turn, discharged it into storage piles over the tunnel and on the ground on each side of the tunnel, the three sizes of stone being kept separate. Sand, hauled by truck from the railroad, was dumped into the receiving end of the portable conveyor unit and was discharged into a storage pile at the outer end of the tunnel beyond the trestle. The sand and crushed stone were discharged through gates in the top of the tunnel and delivered to the concrete mixing plant by belt conveyor and elevator.

Unloading Plant at Railroad Siding.—Bulk material and supplies, principally sand and coal, were unloaded at Powell, on the Hocking Valley Railway, by an unloading plant consisting of a track feeder and bucket elevator and elevated bin from which the material was discharged directly into motor trucks. The equipment was driven by a gasoline engine.

Concrete Mixing Plant.—Practically all the concrete was mixed at a central mixing plant located about 100 ft. up stream from the dam and a short distance east of the east abutment section. The upper part of the plant consisted of four compartments or storage bins for sand and the three sizes of crushed stone; below were two floors above the ground level, the proportioning devices being located on the upper floor and the concrete mixers on the lower. The ground story was a pipe shop.

The equipment consisted principally of two, 1-cu. yd., Smith, tilting mixers; a measuring hopper over each mixer; a collecting hopper, with 2-way chute for feeding the measuring hoppers; four sets of weighing devices, one each for the three sizes of stone and one for the sand; a calibrated water barrel, with 2-way discharge piping; an 8-cu. yd., bottom dump, concrete hopper, in front of the mixers and over the railroad track running under the cableways; and an electrically operated platform elevator for hoisting cement.

Each weighing device consisted of a weigh hopper with a lever-dumping, self-closing bottom; a motor-driven, steel apron conveyor for feeding material to the weigh hopper from an opening in the floor of the storage bin above, from which both were suspended; and a scale-beam with an electric contact so arranged that the lifting of the beam would shut off the conveyor motor. One switch started any one or all of the conveyor motors. The bottoms of the weigh hoppers were connected to a single dumping lever so that, with the scale-beams set, the pressing of the starting switch-button was all that was necessary to weigh out any desired quantities of the three sizes of stone and

of the sand. The pulling of the dumping lever discharged these materials into either of the two measuring hoppers, where the cement was added. The cement was handled direct from and measured as in the original packages.

The specifications required the proportioning of the aggregates by volume. As might be expected, the weight of material for a given volume was not always constant, hence, in order that the materials might be proportioned by weight, which the contractor requested, weighed quantities of the several classes of material were checked, several times during a run, against the volume as measured in the charging hoppers, which were calibrated. If any departure from the required volume was found the scales were correspondingly corrected.

Pumps.—The dewatering of the coffer-dams was done by two, 4-in. discharge, motor-driven, low-lift pumps. When once pumped down the intermittent operation of one pump would handle the leakage. Water for concrete, stone washing, and other plant uses was furnished by two, 3-in. discharge, motor-driven, two-stage centrifugal pumps, one pump being able to supply the demand when the washer was not being operated.

Camp.—The camp included the engineers' office, built for the city by the contractor, as required by the specifications; two contractors' offices; two store buildings; a well equipped emergency hospital, with a graduate nurse in charge day and night; a blacksmith shop; the garage already mentioned; an air-compressor building; a small saw-mill and tool storage building; a large mess hall and commissary; a bath house and latrine; and a varying number of one-story bunk houses, nine being constructed early in the work. All but four of the bunk houses were either torn down or used for other purposes, there being only a few occasions when four were filled. Most of the labor, both skilled and unskilled, drove to the work from the surrounding countryside, or from the dozen or more villages and small towns within a radius of 15 miles, a few making the trip daily from Delaware and Columbus, which were somewhat farther distant.

All the buildings, except the garage and air-compressor buildings, were of frame construction; these two had corrugated iron siding and roofs. The buildings were lighted by electricity and the bunk houses, latrine, and commissary were well ventilated and screened. The engineers' office was the best equipped building in the camp, being provided with a furnace, Delco lighting and power outfit, a drilled well, motor-driven pump, a well-equipped wash and toilet room, and a fireproof concrete vault for the storage of note-books, drawings, and other records.

Sanitation.—Camp wastes were handled in several ways. The large latrine, and smaller ones at several points on the work, and the mess-hall kitchen were provided with galvanized cans which could be removed, through doors at ground level, from the outside of the buildings. All wastes so collected were buried in pits dug at some distance from the work, the contents of the pits being covered with quicklime and a few inches of earth as soon as deposited. Wash water from the latrine and kitchen was run into farm tile drains from which it was allowed, by sub-surface irrigation, to leach into the ground. The engineers' office was provided with a small sewage treatment plant, consisting

of a covered circular concrete septic tank and a sub-surface irrigation system of two units. Each unit had a length of 232 ft. of vitrified pipe, wrapped with burlap at the joints and surrounded with crushed stone. Drinking water, other than that for the engineers' office, was obtained from a 100-ft. well by a motor-driven pump and stored in an elevated wooden tank erected close to the mess hall to which it was piped.

Other Plant and Equipment.—Mention should also be made of the cement storage house, 60 ft. wide by 100 ft. long, with a capacity of about twenty carloads of cement, a stiff-leg derrick with an 80-ft. boom, used for many purposes and at several points on the work, the road-building machinery, a complete compressed-air operated drill dressing outfit, and a grouting machine.

Coffer-Dams

In designing the dam it was obviously desirable to take full advantage of the topography of the top surface of the rock, as developed by the test borings, and to make the spillway as long as possible, in order to keep at a minimum the depth of water which would have to pass over the spillway under the assumed maximum flood flow. It was decided, therefore, to place the ends of the spillway section at about the points where the rock surface reached the elevation of the crest of the spillway. This brought the center of the spillway close to the low-water mark on the east side of the existing river bed, at which point the rock outcropped and there was no question, either from surface indications or borings, as to the suitability of the location for the outlet gate-house.

As previously stated, provision was made in the plans for placing four 48-in. pipes in the up-stream part of the gate-chamber to take care of the ordinary flow of the river during the construction period. The construction of the gate-house was, therefore, the first work started in the river bed. The first coffer-dam, on the east side of the river, was begun in the early part of September, 1922, before the completion of the general construction plant. It was planned by the contractor to extend the coffer-dam to Elevation 790, 9 ft. above low water, but on the recommendation of the engineers the height was reduced to about Elevation 784, roughly 3 ft. above low water in the river, as coffer-dams of this height had proved satisfactory in the construction of the Julian Griggs Dam.

In addition to that section of the dam in which the outlet gate-house is located, sufficient area for the next section to the west, out into the river, was included in the east coffer-dam, the inside dimensions of which were 175 ft., measured at right angles, and between 145 and 150 ft., measured in a direction parallel to the dam. For the construction of the up-stream, down-stream, and river sides of the coffer-dam, cribs, about 10 ft. in width, made of small logs secured on the site of the work, were built, the framing being done on the bank of the river. When complete, the cribs were skidded into the river, floated into position, fastened together, and sunk by filling them with rock. Earth was then deposited around the outside of the coffer-dam, after which 2-in. sheeting was placed, edge to edge, all the way around the outside face of the cribbing. The sheeting was held at the top by waling timbers

attached to the cribbing, and was driven into the earth by mauls. As the water was pumped down in the coffer-dam additional earth and small rock was placed in the cribbing and outside the sheeting, the outside fill being compacted by men, with rubber boots, until the top of the fill was above the water level in the river. In spite of the fact that the entire easterly part of the river bed was bare ledge rock, a remarkably tight contact was secured.

The first construction work in the coffer-dam was started on October 6, 1922, and continued until December 28, when the coffer-dam was topped by a moderate flood. An attempt was made to pump the water out during the early part of the following month, but this had to be abandoned on account of another rise in the river. It was then decided to cease operations until spring. Some trouble was encountered during the winter with ice jams, but only one actually did any damage. A view of the east coffer-dam, taken on November 29, 1922, is given in Fig. 22.

On May 1, 1923, the coffer-dam was pumped out and construction work resumed, but it was again flooded on May 12, and it was not until May 24 that construction work was again started. On June 6, and again on June 15, heavy local showers raised the river sufficiently to flood the work for an hour or two, but operations were resumed the following day.

On June 25, 1923, an opening was made in the east coffer-dam and the river turned into the 48-in. pipes passing through the gate-chamber. On the same day the up-stream portion of the coffer-dam on the west side of the river, previously constructed, was closed. The up-stream side of the west coffer-dam was similar in construction to the east coffer-dam, but was tied into a concrete wall, constructed for the purpose, which projected up stream several feet from the westerly end of the concrete already in place in the east coffer-dam. The river side of the west coffer-dam was also of the same construction, the cribbing resting on the concrete apron, which was already in place, and tying into the westerly end of the down-stream cribbing of the east coffer-dam. The down-stream side of the west coffer-dam consisted of an earth fill made of earth excavated from the west bank of the river. The up-stream side of the west coffer-dam was also reinforced by dumping excavated material on the river side of the cribbing, raising the effective height to about Elevation 788. The area enclosed by the west coffer-dam was, roughly, 130 ft., up and down stream, by 180 ft. parallel to the axis of the dam.

The west coffer-dam was flooded twice, first, on July 8, 1923, when a part of the down-stream fill was washed out and had to be replaced, and the second time on September 22, when little damage was done. On November 2, 1923, the concrete work was completed to a point above low-water level. From this time, however, the up-stream side of the coffer-dam continued to be used as a means of crossing the river, a timber foot-bridge spanning the opening through which the river flow passed to the 48-in. pipes in the gate-chamber.

Excavation

Before describing the methods used in making the excavation for the dam and appurtenant work it will be well to state briefly the conditions existing at the site. The Columbus or Delaware limestone was encountered at all

points under the dam and core walls, the cover varying, however, both in character and thickness, the latter reaching a maximum of about 15 ft.

The higher lying banks outside of and at each end of the concrete part of the dam were cultivated areas with 2 to 12 ft. of cover consisting of clay loam, yellow boulder clay, and a varying thickness of clay mixed with rock fragments immediately above the ledge rock. The first sharp rise from the river bed on the east bank had only such cover as was held in place by the roots of trees, brush, and weeds. East of this rise was a moderately sloping terrace, the rock being covered with from 2 to 5 ft. of leaf mould, loam, and clay and rock fragments mixed. In this area the top of the rock had been affected by weathering to a greater extent than at any other part of the site except for a few small areas on the west bank. The second sharp rise east of the river was covered with material similar to that overlying the terrace, but of less depth.

The easterly half of the river channel was bare ledge rock, whereas the westerly half was covered to a depth of from 1 to 4 ft. with a bar of dirty sand, gravel, and small fragments of rock. Immediately west of the river was an alluvial terrace from 30 to 50 ft. wide, the cover over the rock running from 4 to 9 ft. in thickness and of material similar to that found in the river, except with a considerable admixture of leaf mould and loam. The lower two-thirds of the steep slope west of the river consisted of rock steps filled, to an evenly sloping top surface, with loam, leaf mould, and clay and rock fragments of all sizes, some measuring as much as 5 or 6 ft. in one dimension. Some of the vertical faces of the steps were 15 ft. high and both the vertical and horizontal faces were more or less weathered, in a few places, to a considerable depth.

It is interesting to note that the conditions revealed during excavation in this area on the west side of the river were not disclosed by the preliminary investigations. As a result, the foundations of the dam, the spillway channels, and the concrete aprons of all but one section west of the river, were carried to lower elevations than originally contemplated.

The upper third of the west slope had scarcely any cover over the rock but, like the entire area between the ends of the spillway, except the river bed, was covered with trees, brush, and weeds.

The excavation for the dam and appurtenant work was divided into four classes: Earth excavation above Elevation 782; rock excavation above Elevation 782; excavation below Elevation 782; and rock excavation in the cut-off trench. The contract prices for the first three classes of excavation were the same, $1.90 per cu. yd., and, for rock excavation in the cut-off trench, $12.00 per cu. yd.

Except for some hand work for the engineers' office and certain small culverts, actual excavation was started on July 22, 1922, on that part of the relocated highway east of the dam where cuts were necessary. This was followed by the stripping of the cover and loose top rock on the terrace east of the river, all the excavated material from these two locations being placed in embankments made necessary by the relocated highway. Excavation for the

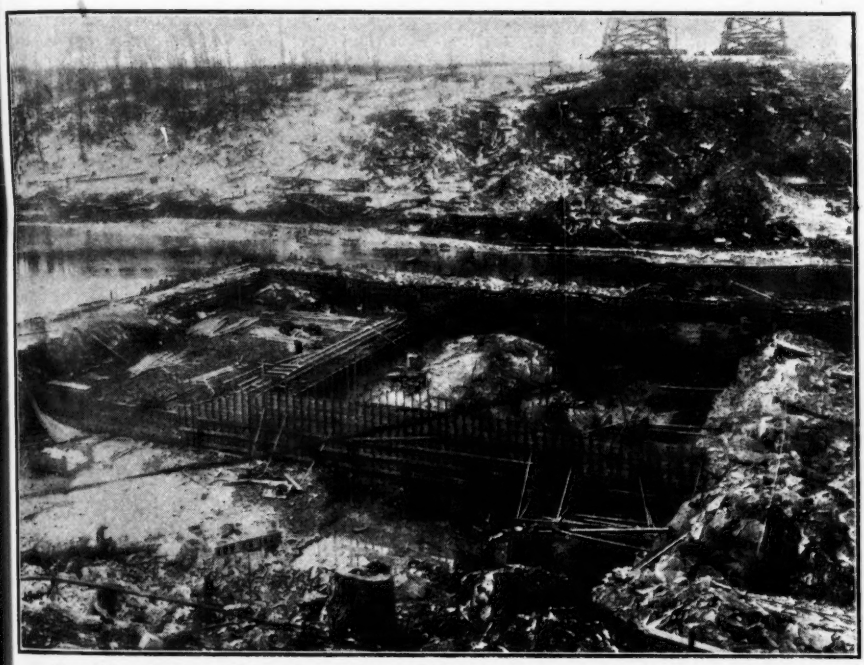

FIG. 22.—VIEW OF EAST COFFER-DAM, O'SHAUGHNESSY DAM, NOVEMBER 29, 1922.

FIG. 23.—RATHBONE BRIDGE—THE OLD AND THE NEW.

east section of the spillway and rough excavation for the east spillway channel came next, followed, successively, by excavation for the sections of dam and spillway channels west of the easterly section, as far as the top of the first steep slope east of the river. Some of the excavated material was used to complete the road embankments, the remainder being placed in storage piles adjacent to the stone crushers up to the time the crushers went into operation. Prior to the completion of the first cableway (there were two), all the hauling of excavated material was done by trucks, and thereafter by both cableway and trucks, the trucks hauling part of the excavated rock direct to the crushing plant and a large part of the remainder to large steel skips which, in turn, were handled by the cableways.

On October 10, 1922, excavation by steam shovel was started in the first coffer-dam, and completed on November 15. On this latter date rough excavation for the remaining sections of the dam and spillway channels east of the outlet gate-house was started. This work was carried on, as weather permitted, during the winter, all the rough excavation for the remaining sections of the dam, east of the river, being completed, as well as a part of the spillway channel excavation, by March 27, 1923, when the shovel was moved to the west side of the river and excavation started for the westerly spillway section.

Excavation on the west bank was carried on, successively, from rock step to rock step, working toward the river, part of the excavated rock being taken to the crushing plant, by cableway and railroad, the remainder being cast over on the next lower section and again rehandled. It was during this period that a second steam shovel was put in operation, on the terrace just west of the river.

On July 14, 1923, steam shovel excavation was begun in the west coffer-dam, following which came the completion of the excavation on the terrace immediately west of the river. Part of the rock from the coffer-dam was transported to the crushing plant, but the bulk of the mixed earth and rock from the west bank was placed in spoil banks below Elevation 792 on the upstream side of the dam. By the early part of September the steam shovel work for the dam proper was complete, and work was again taken up on the spillway channels. It was necessary to go over the spillway channels at least once and, in some instances, several times to put them in final acceptable condition, this work being partly done during the remainder of 1923 and completed in 1924. The excavation necessary for the new road connecting the west end of the dam with the existing county highways, as well as the excavation of most of the earth used in building the embankment sections of the dam, was made during the latter part of 1924.

A considerable amount of hand excavation was necessary in trimming the area under the dam and apron and the spillway channels. The bulk of the core wall and cut-off trench excavation was hand work supplemented by light blasting of the rock, the sides of the cut-off trench first having been cut with a channeling machine to the full depth. The excavation for the pavilion walls and for the approach channel to and for the head-race below the power gate-house was also done by hand mucking after blasting.

Throughout the work it was necessary to drill and blast the rock before it could be handled by steam shovel, and wherever the excavation had to be brought to neat lines it was necessary to line drill the rock before blasting to prevent overbreaks and to hand trim the faces of the excavation, this being especially the case along the edges of the spillway channels where, even with the precautions noted, some overbreaks occurred. Practically all the rock excavated by hand was handled by the cableways where the work was within the area served by them, and by truck from all points outside this area.

Grouting the Foundation

The information obtained from the core borings made on the site of the dam and from the hydraulic tests made on the cored holes resulted in the decision to grout the foundation in order to close up any open bedding planes, solution channels, cleavage cracks, or fissures that had been located by the borings or tests, or that might exist at points not explored by the borings. The hydraulic tests had further developed the fact that in general the rock was fairly water-tight below a depth of approximately 10 ft. or less below the top of ledge rock, except at a few points which could be given special attention. It was decided, therefore, that in general the grout holes should be drilled to an approximate depth of from 8 to 10 ft. below the top of the rock, after the excavation was completed. The depth, however, was varied by drilling deeper along the edges of the steps in the foundation adjacent to the lower sections of the dam and to a less depth on the other side of the steps, the holes along the side adjacent to a lower section reaching to from 3 to 6 ft. below the top of the rock in the lower section after the excavation in this section had been completed. It was also decided that there should be a row of holes up stream from the cut-off trench, unless the rock at this location showed no seams or cracks, and a row in the cut-off trench, the holes in these rows to be spaced not more than 12 ft. apart, and also holes on 10 to 12-ft. centers, both ways, down stream from the cut-off trench and under the dam, but not under the apron, and additional holes into every crack and crevice found in the excavated area.

A number of crevices were opened by the excavation, a few at the easterly end of the spillway section and just west of the river bed being filled with earth. Each crevice was cleaned out with water under pressure and grout pipes were placed in the crevice, or holes drilled, so as to intercept the crevice, or both, with the idea of making it possible for the grout to reach every possible channel through which water might seep through under the dam. In several of the spillway sections no attempt was made to follow any regular spacing of holes, the location of the cracks and seams in the rock determining the location and spacing of the holes drilled.

A total of 553 grout holes were drilled under the dam, abutments, and core walls, 526 of this number being drilled with jack-hammers, using a 2-in. bit, the over-run in width at the top of the hole being enough to admit the 2-in. standard weight, wrought-steel pipe used for grouting. These holes ranged from a minimum of 4.1 ft. to a maximum of 11.1 ft. in depth, the average being 9.6 ft. For the holes under the large concrete abutment blocks at either

end of the spillway and under the two westerly sections of the spillway, it was necessary to use a well-drilling machine with a 4-in. spud-bit to reach to a depth lower than the bottom of the adjacent section of the dam. Twenty-seven machine-drilled holes were put down, ranging from 18.0 to 30.5 ft. in depth, the average depth being 26.4 ft.

As soon as the drilling was completed, grout pipes of sufficient length to reach above the first lift of concrete were put in place. In the holes drilled with jack-hammers, 2-in. pipe was used; in the machine-drilled holes, 4-in. pipe was required, the pipe being cemented in, where necessary, to exclude the concrete. Where the elevations of the foundation of adjacent sections of the spillway differed by more than about 8 ft., it was necessary to hold back the grouting of the higher section until part of the concrete had been placed in the lower section, pipe being extended up through any additional lifts of concrete placed in the higher section. In some of the sections at the westerly end of the spillway the pipes were carried through the up-stream face of the dam. In the sections of the spillway in the river bed, it became necessary to limit the length of pipe for which payment would be allowed. The extension of the pipe was permitted, however, in order to expedite the placing of concrete; but the length for which payment was allowed was limited to 4 to 5 ft. under the dam and 11 to 14 ft. in the cut-off trench. For each fitting that was necessary, an allowance of 1 lin. ft. of pipe was made and the figures given subsequently include this allowance.

Excluding the grout-pipe connections to the 48-in. outlet pipes in the gate-chamber, and pipe used for other purposes, a total of 4 363 lin. ft. of pipe was paid for, the minimum length for any one hole being 2 ft. and the maximum, 43 ft. The total length included eight grout-pipe connections to sink holes, springs, and mud seams where no drilled holes were required.

The grouting machine was made on the job from a stock cylindrical steel air reservoir, 28 in. in diameter by 44 in. high, having pressed oval heads, the upper head containing an oval cast-iron clean-out manhole held in place by two pressed steel yokes and bolts. Angle-iron legs were welded to the tank, and a 3-in. outlet pipe, with a 3-in. brass cock and reducer for connecting to 2-in. pressure hose, was screwed into the bottom head. For mixing the grout four ½-in. pipes were screwed through the lower head about 4 in. from the outside shell and on the quarter-points, each pipe having an elbow and short nipple on the inside end and all four pipes being connected to a 1-in. air line running up the outside of the tank. This air line had a 3-way cock, to the outstanding opening of which the air-supply hose was connected, and two tees, one for the pressure-gauge connection, and the other a blow-off, with a valve, which extended through the top head of the tank.

The operation of mixing the grout consisted of filling the tank to the proper point with water, through the manhole, turning in a small quantity of air through the mixing pipes, placing 2 sacks of cement and 2 sacks of sand in the tank, screwing the manhole casting in place, the blow-off being open, and turning on of full air supply to the mixing pipes for at least 1 min.

Connection between the tank and the grout pipes was made with 2-in., rubber-lined linen hose. The grout was applied by closing the blow-off valve

and opening the outlet cock, the air supply being turned in through the top connection if it failed to move the grout when being supplied through the mixing pipes.

Payment for the grout placed was made by the cubic yard which by definition consisted of 8 cu. ft. of cement, 8 cu. ft. of sand, and the necessary water, the actual volume of this quantity of wet grout being somewhat less than 1 cu. yd. A fine brick sand, 45% or more of which would pass a 40-mesh screen, was used. A total of 229 cu. yd. of grout was placed, the quantity in the several sections of the spillway varying widely. The minimum quantity placed in any one section, 71 ft. 6 in. in length measured along the axis of the dam, was 0.75 cu. yd. The maximum quantity in any one section was 38.63 cu. yd.; and in one hole in this section, 18 cu. yd. were placed, this quantity being the maximum for any one hole. The average quantity per hole, including the sink holes, springs, and mud seams, was 0.41 cu. yd.

The full air pressure carried in the supply lines, of 80 to 100 lb. per sq. in., was applied to the holes, only 50 lb., however, being required by the specifications. If grout rose in any of the holes surrounding the one being grouted, the pipes were capped and the grouting was continued. In one case grout was raised in fifteen holes from one connection. Later, the caps on the pipes were removed; if they were not filled, grout was then pumped into them, the same treatment being applied to all pipes that had taken grout upon the first connection.

The number and length of drilled holes, the amount of pipe and fittings used, and the grout placed considerably exceeded the estimated quantities, but the results obtained have been considered as amply warranting the added cost. A thorough examination of the area down stream from the dam has failed to disclose any leakage other than a slight seepage around the three westerly sections. This leakage is so small as to cause no uneasiness from the possible formation of solution channels through the rock.

Concrete

In the construction of the dam, five classes of concrete were used. For convenience of reference Table 1 gives the mixture used in, the contract price per cubic yard of, the volume placed under, and the location in the work of, each class of concrete. The total volume of concrete placed was 79 565 cu. yd., at an average price of $9.29 per cu. yd.

The specifications permitted the engineer to change the relative volumes of the fine and coarse aggregates in each mixture, but at the same time keeping the total volume of the two classes of aggregate the same. The coarse aggregate was screened into three sizes: No. 1, ¼ to ¾ in.; No. 2, ¾ to 1½ in.; and No. 3, 1½ to 3 in. In most of the work a mixture of the three sizes was used, but in much of the reinforced work the No. 3 coarse aggregate was omitted, and, at times, only the No. 1 coarse aggregate was used, especially in thin reinforced work, in order to ensure satisfactory bedding of the steel reinforcement bars.

The specifications for the fine aggregate provided that the material should contain not less than 80% of silica, graded from fine to coarse, should be

thoroughly screened to reject all particles greater than ¼ in. in diameter, and should not contain more than 3% of material passing a 100-mesh standard sieve; also, that the material should not contain more than 3% of loam, clay, or dirt. The fine aggregate used was a washed bank sand shipped from Dresden, Ohio, distant about 55 miles east of Columbus.

TABLE 1.—CONCRETE USED IN THE O'SHAUGHNESSY DAM.

Class.	Mixture.	Contract price per cubic yard.	Volume placed, in cubic yards.	Location.
A.....	1:3 :6*	$ 6.85	56 189	Cyclopean masonry forming the body of the dam and bridge piers.
B.....	1:2½:5	13.00	2 032	Core walls of the earth dikes; retaining walls and culverts.
C.....	1:2 :4	14.00	17 362	Exposed faces of dam and bridge piers; cut-off trench; substructures, not including floors of gate houses and of pavilion; paving and retaining walls in regulating works below dam.
D.....	1:2 :4	21.00	4 002	Bridge arches, spandrel and curtain-walls; floor of bridge, including sidewalks; floor of gate-chambers, pavilion, and balconies; and bulkheads in power tubes.
E.....	1:2 :4	40.00	34	Roof decks for superstructures of gate-houses and pavilion.
Average.....	.....	$ 9.29	.....	.....
Total.....	.....	.....	79 569	....

* Not including the embedded stones.

The coarse aggregate was crushed limestone obtained mainly from the rock excavation for the dam and spillway channels and was screened on the job into the three sizes mentioned.

In the 1 : 3 : 6 Class A concrete, designated as cyclopean masonry and forming the main body of the dam and piers of the bridge over the spillway, large stones, from the rock excavation, were embedded to a certain extent, but the percentage of such stones, taken as a whole, was small.

Alpha Portland cement was used throughout the work, the total amount being 93 500 bbl. With a total volume of 79 569 cu. yd. of concrete placed in the dam, the ratio of cement to concrete was 1.18 bbl. of cement per cubic yard of concrete.

The proportioning of the relative volumes of fine and coarse aggregate and the proportioning of the three sizes of coarse aggregate was adjusted throughout the course of the work with the object of securing concrete of maximum density.

In all the concrete very little departure, in general, was made from the specified proportions of fine and coarse aggregate. When three sizes of coarse aggregate were used, the proportions were, in general, 1 volume of No. 1, 1 volume of No. 2, and 2 volumes of No. 3 aggregate, and when only two sizes were used the proportions were, generally, 1 volume of No. 1 and 2 volumes of No. 2.

Expansion Joints

In the spillway section of the dam, under each arch of the bridge, an expansion joint was located. At each of these expansion joints, in addition to the so-called bonding grooves, Fig. 14, a copper strip, ⅛ in. thick and 7 in. wide, was inserted across the down-stream side of the up-stream bonding groove. The copper strip was so placed as to be free to move with the contraction, and subsequent expansion, of the adjacent sections of the dam.

The concrete roadway slab, carrying the brick pavement, and the concrete sidewalks had expansion joints at the third points of each arch span, at the center lines of the walls forming the upper extension of the bridge piers, and at suitably placed locations opposite the gate-houses and pavilion. In the sidewalks and curbs, Carey "Elastite" was used in the expansion joints.

Expansion joints were also provided in the Indiana limestone balustrade on the bridge as close to all sidewalk joints as the design would permit, no mortar being placed between the stones at these points, the upper rail being held by 1-in., brass dowel pins, 6 in. long.

Construction of Earth Approaches

At each end of the spillway section the approaches were built of earth embankments, with a core wall in the center of each. In the construction of the embankments the surface of the ground was first stripped and all perishable matter removed, as well as loose stones and boulders. The embankments were made of yellow boulder clay, obtained from borrow-pits, and were free from perishable matter and from stones of more than 3 in. in their greatest dimension. On the up-stream side of the embankments, selected material was used which would compact and be as nearly water-tight as possible. On the down-stream side the requirements as to the materials were less exacting and in certain parts some small stone was placed near the outer face, but not in pockets, and back from the face to allow a sufficient thickness of earth covering. Frozen materials were not permitted to be used, nor were embankments built in freezing weather.

The materials were spread in approximately horizontal layers not exceeding 4 in. in thickness and were compacted by rolling with power-operated grooved rollers weighing not less than 2 000 lb. per lin. ft. of width and not less than 4 ft. wide. If the materials were not sufficiently moist when spread, they were moistened with water so as to permit of proper compacting.

The core walls in the approaches began at the outside of the bridge abutments and extended away from each bridge abutment to a point 20 ft. beyond the point where Elevation 858.5 met the original surface of the ground. They were carried down 12 in. into solid rock.

The bituminous macadam roadways on the approaches connect with the bridge over the spillway and with the relocated roads built to connect with existing county highways. A typical section through one of the earth approaches is shown in Fig. 18.

The Reservoir

The O'Shaughnessy and Julian Griggs Reservoirs are quite similar in general characteristics, in that both are long and narrow, the main differences being that the O'Shaughnessy Reservoir is deeper, somewhat longer, and impounds about three times as much water.

The total length of the O'Shaughnessy Reservoir is 8.0 miles. At Elevation 845, normal water level, the reservoir has an average width of 855 ft.; the maximum width is about 1 900 ft., about 0.4 mile above the dam; the minimum width is about 200 ft., at Bellepoint, where the river widens and forms the upper end of the reservoir.

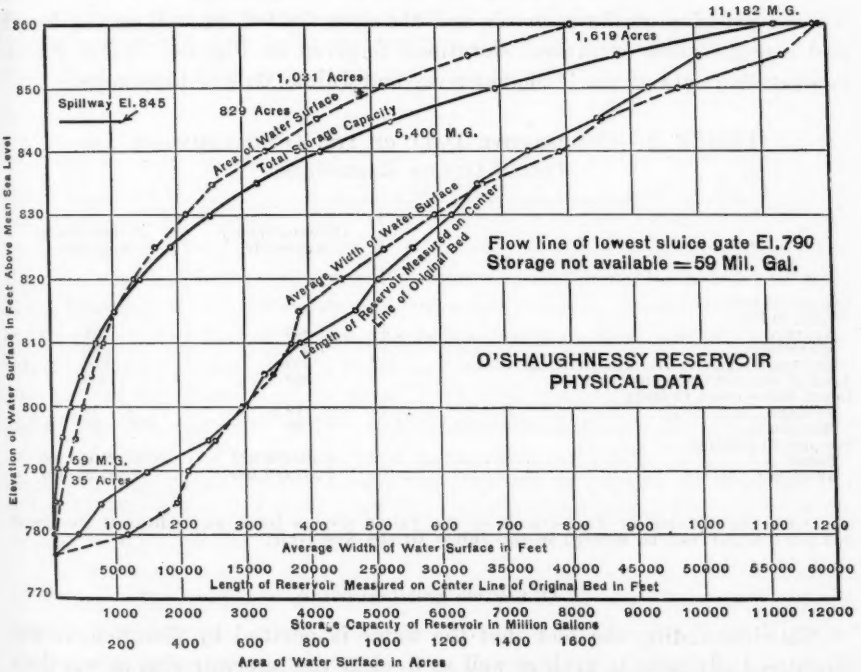
11,182 M.G.
1,619 Acres
1,031 Acres
Spillway El. 845
829 Acres
Area of Water Surface
Total Storage Capacity
5,400 M.G.
Average Width of Water Surface
Length of Reservoir Measured on Center Line of Original Bed
Flow line of lowest sluice gate El. 790
Storage not available = 59 Mil. Gal.
O'SHAUGHNESSY RESERVOIR
PHYSICAL DATA
59 M.G.
35 Acres
Elevation of Water Surface in Feet Above Mean Sea Level
Average Width of Water Surface in Feet
Length of Reservoir Measured on Center Line of Original Bed in Feet
Storage Capacity of Reservoir in Million Gallons
Area of Water Surface in Acres

FIG. 24.—O'SHAUGHNESSY RESERVOIR, CAPACITY, AREA FLOODED, LENGTH, AND AVERAGE WIDTH AT VARIOUS ELEVATIONS.

The area of the water surface at Elevation 845 is 829 acres, or 1.57 sq. miles. For sanitary, as well as for general control, a wide marginal strip was acquired by the City on each side of the reservoir. On the west side, this strip has an average width of 227 ft.; on the east side, the average width is 278 ft.; the greater width on the east side being due to the fact that the City acquired all the land between the reservoir and a new highway built to replace one which is now submerged. On the west side the marginal strip has an area of 204 acres, and on the east side, 241 acres, the total area of the marginal strip on both sides of the reservoir being 445 acres, or equal to about 54% of the area of the water surface of the reservoir. The total area of the reservoir and the marginal strip combined, is 1 274 acres, or about 1.99 sq. miles.

On the west side of the reservoir, about 2.5 miles above the dam, at the Girls Industrial School, a correctional institution of the State of Ohio, and for a distance of about 3 200 ft., no marginal strip was acquired, as the land is owned by the State and the City lacked the power to acquire the property except by purchase, and the State did not care to sell. The City, however, did secure the right to flood the State-owned land, along the river, so as to be able to construct the reservoir.

The reservoir has an average depth of 19.8 ft., the maximum being 69 ft., just above the dam. The total volume impounded at Elevation 845 is 5 400 000 000 gal.; of this, 5 341 000 000 gal. are available, taking low water at Elevation 790, or 55 ft. below the crest of the spillway.

The capacity of the reservoir and the area flooded, as well as the length and average width at various elevations, is given on Fig. 24. Table 2 gives comparative data of the O'Shaughnessy and Julian Griggs Reservoirs.

TABLE 2.—COMPARATIVE DATA OF THE O'SHAUGHNESSY AND JULIAN GRIGGS RESERVOIRS.

	O'Shaughnessy Reservoir.	Julian Griggs Reservoir.
Length, in miles	8.0	5.8
Width, in feet:		
Average	855	516
Maximum	1 900	820
Minimum	200	200
Area of water surface, in acres	829	363
Depth below crest, in feet:		
Average	19.8	14.5
Maximum	69	85
Storage, in gallons*:		
Total	5 400 000 000	1 720 000 000
Available	5 341 000 000	1 487 000 000

* By the addition of flash-boards to the Julian Griggs Dam, an additional storage of 500 000 000 gal. can be secured in the Julian Griggs Reservoir.

CLEARING AND GRUBBING

Notwithstanding the fact that the water is purified by filtration, it was considered advisable to grub as well as to clear the reservoir site, as was done in the case of the Julian Griggs Reservoir. Clearing and grubbing, therefore, was carried out over the whole of the area flooded and up to an elevation 2 ft. above normal water surface, the total area so cleared and grubbed being 741 acres.

All trees, saplings, and brush were cut down flush with the surface of the ground, and all stumps and roots larger than 2 in. in diameter were removed to a depth of at least 6 in. below the surface of the ground. In removing stumps both dynamite and stump-pullers were used. Prior to clearing and grubbing, property owners from whom land was acquired, were permitted to cut down and remove such of the timber as they desired.

On the west side of the reservoir the property acquired by the City was not bounded by a highway, and line fences and stone walls were left in place, the City permitting abutting owners, for the present, to water stock at the

reservoir edge. On the east side of the reservoir, where the land acquired is bounded by a new highway, all fences were removed and stone walls leveled. All material removed in the clearing and grubbing operations became the property of the contractor and was either burned or removed to points outside the limits of the property owned by the City. Material was not disposed of by throwing it into the river and allowing it to float down stream. After the clearing and grubbing had been completed, the entire ground surface was burned over, and, prior to filling the reservoir, where grass, weeds, and vegetation had grown up subsequent to the first burning over, the ground was burned over a second time.

The removal of buildings was not included as part of the work of clearing and grubbing, arrangements having been made with some property owners for the removal of their buildings, whereas the other buildings were sold, the prices paid by the purchasers including their removal.

Relocation of Roads

From the dam to the upper end of the reservoir, on the east side of the reservoir, there existed county roads running, in general, parallel to the river. For much of the distance these roads were at such an elevation that they would be flooded when the reservoir was filled, and it was incumbent on the City, therefore, to build a new road to replace those flooded. On the west side of the reservoir it was necessary to build a short stretch of new road where an existing road was flooded, above and below Eversole Run, one of the tributaries of the river, and also to build a second short stretch of road from the west end of the dam to an existing north and south highway. At the east end of the dam, the earth section started, practically, at the new road. Where new roads were built arrangements were made with the County Commissioners of Delaware County whereby the City was to acquire the land for and build the new roads and then deed them to the County, the County, in turn, to deed to the City the old roads which would be flooded.

The old roads were mainly gravel roads, in some places nearly impassable in winter. In replacing these roads the City built water-bound macadam highways conforming, in general, to the standards of the Ohio State Highway Department for similar conditions. The macadam surface was 16 ft. in width and had a finished thickness, after rolling, of 6½ in. Crushed limestone, obtained locally, was used. With the exception of the Rathbone Bridge, mentioned later, and the bridge over the dam, the macadam surfacing was carried continuously over the reinforced concrete bridges spanning the small streams tributary to the reservoir. A total length of 7.5 miles of new road was built by the City. In 1926, the new road on the east side of the reservoir was taken over by the State of Ohio and incorporated as a part of the State Highway System, and it has been improved by widening the surfacing and making it a bituminous-surfaced, water-bound macadam.

Reconstruction of Bridges

Mention has already been made of the replacement of the Powell Bridge by the reinforced concrete arch bridge over the dam. At the Girls Industrial

School, about 3.1 miles above the dam, the river was spanned by a bridge known as the Rathbone Bridge. With the filling of the reservoir this bridge would have been entirely submerged. As the bridge was on a main east and west highway, a crossing of the reservoir at this point could not be abandoned.

The superstructure of the existing bridge consisted of three 100-ft. steel trusses. These trusses were in good condition and were suitable for use in a new bridge, if reinforced. Wishing to make use of these trusses in the new bridge if economically possible, the specifications therefore were drawn so as to permit a bidder to submit a proposal for raising the three trusses a total height of about 35 ft. and of moving them on to new piers, and then lengthening the bridge by the addition of eight new 100-ft. trusses of similar construction, or of building an entirely new bridge.

The low bid received, on the basis of using the three old trusses, was $119 240. The low bid for an entirely new bridge, with 16-ft. roadway, was $115 968, and $120 718 with an 18-ft. roadway. The City accepted the bid for an entirely new bridge, with the 18-ft. roadway.

The superstructure of the new Rathbone Bridge consists of five steel trusses, each 207 ft. 6¾ in. in length. The piers and abutments are of concrete. The bridge has a total over-all length of 1 040 ft., face to face of end abutments, and it is the longest bridge in Delaware County. The lower clearance line for the bottom chord and floor members is 13 ft. 6 in. above normal water level in the reservoir. The entire structure was built in accordance with the requirements of the 1918 Specifications for Highway Structures, Class T-15, of the Ohio State Highway Department. A view of the old bridge and the new is shown in Fig. 23.

On the west side of the reservoir at Eversole Run, previously mentioned, a small reinforced concrete bridge was built in connection with the relocation of a part of one of the county roads, and on the east side of the reservoir along the new road several small culvert bridges were built.

Moving Church and Cemetery

Near the upper end of the reservoir was the Scioto Valley Christian Union Church and the Oller Cemetery. The church and cemetery served a rural population and had been there for years. With the flow line adopted for the reservoir, it was found that a part of the cemetery would be submerged and that the church would either have to be moved or a new church built.

Following a series of conferences with the Church Trustees it was decided to move the church. This was accordingly done, the City acquiring and deeding to the Church Trustees the new piece of property on which the church is relocated. The church was a small brick building, 31 ft. wide by 39 ft. long, with 8-in. walls; it was moved horizontally a distance of 350 ft. and, at the same time, raised a distance of 14.2 ft., vertically. To avoid cracking the walls, while being moved, a heavy steel band was placed around the building.

When it comes to moving a cemetery, however small, the question of sentiment has to be taken into account. A happy solution of the problem, however, was found when the Cemetery Trustees agreed to supervise and do all the

work, the City paying all the bills for labor and material. All told, 99 re-interments were made.

Reconstruction at Girls Industrial School

At the Girls Industrial School, a very considerable amount of reconstruction work was necessary. In fact, barring the construction of the dam proper, it was the one most expensive item of construction involved in building the reservoir. The institution, in addition to housing the necessary State officials and employees, houses between 600 and 700 inmates, and for years had been in its present location on the west bank of the river. It had been expanded from time to time and new utilities and conveniences added, some of which were not far from, and only slightly above, the river. The best site for the dam was down stream from the school, but the adoption of that site involved flooding State property, buildings, and utilities. The City had no power to condemn State property, so it was faced with the situation of either securing permission from the State to flood some of the property owned by the State, and replace the buildings and utilities which would be wholly or partly submerged, or of constructing the dam on a very much less desirable site above the school. The City decided to seek permission from the State to flood State property and bear the necessary cost of replacements. To secure this permission it was necessary to secure an Act of the Legislature. Such an Act was passed, putting the matter, in so far as the State was concerned, in the hands of the Department of Public Welfare and authorizing the Director of Public Welfare to make the necessary arrangements with the City. The replacements made by the City included the building of a new power house, laundry, and sewage disposal works, together with the necessary connections thereto, and, after these replacements were made, they were turned over to and became the property of the State.

Power House.—The power house was a short distance from the river and supplied the institution with light, heat, and water. The building was old and in need of repair, and some of the equipment was only in fair condition. The site had been poorly chosen, as the power house had been flooded in the past with high water in the river. The City built for the institution a new brick power house at a much higher and more convenient location.

Such of the existing equipment as could be moved, was transported to the new building and put in running order. The principal items of equipment moved were two 200-h.p. boilers (new brick settings had to be provided), two feed-water heaters, each 6 ft. in diameter by 16 ft. high, two 100-kw. and one 75-kw. engine generator sets, one 5-panel switchboard and appurtenances, one 400 gal. per min., duplex steam pump, two boiler feed pumps, a hot-water pump, a boiler-room scale, and a rapid sand-filter plant of the wooden-tub type; also, such minor equipment and piping as could be used to advantage. While the equipment was being moved, the institution had to be kept running, and at no time was there any interruption in the service of supplying light, heat, and water to the buildings. It was necessary, however, for the City to furnish and install in the new power house some new equipment and material. This consisted mainly of two 200-h.p. boilers and a 400 gal. per min. duplex

steam pump for pumping filtered water, as well as some new piping, etc. In connection with the new power house, the City also built a reinforced concrete coal-pocket opening direct from the boiler room.

After the new power house was in regular service, the old power house was abandoned, the building wrecked, and the chimney dynamited.

Water Supply.—The institution had two sources of water supply, one, from a sulfur-water well for drinking purposes, and the other, filtered river water for general use. With the filling of the reservoir, the sulfur-water well was flooded, so the City provided a new drinking-water supply by drilling a well located inside the new power house. The river water supply was not so simple. With the old power house, located near the river, the raw-water pumps could take water direct from the river without an excessive suction lift, but with the new power house, at a much higher elevation and at a distance of about 550 ft. from the river, it was too expensive to build a suction well about 60 ft. deep and run a low-level suction conduit to the river so that water could be obtained with the reservoir empty, a condition which would be the same as if no reservoir existed.

The solution adopted for supplying raw water was to build an electrically operated raw-water pumping station out in the reservoir and to run duplicate cast-iron force mains from this station to the power house. The substructure of the pumping station was obtained by extending, down stream, one of the concrete piers of the new Rathbone Bridge, the pier being one located in the old river bed, and constructing in it a suction well and two dry pump wells, each pump well being equipped with a horizontal electrically operated centrifugal pump, automatically and manually controlled from the power house, 700 ft. away. Each pump has a capacity of 500 gal. per min. against a normal head of 42 ft. and a maximum head of 90 ft.

The superstructure of the pumping station is a small brick building, with the floor at practically the same elevation as the floor of the bridge. Access to the building is direct from the bridge floor. At the power house the City built a small reinforced concrete coagulating basin, and also a small filtered-water reservoir of similar construction.

Laundry.—The laundry for the institution was built on somewhat higher ground than the power house, but still not sufficiently high to have escaped partial submergence with the reservoir full. The building was of brick and consisted of three wings, connected by narrow galleries, and had a total floor space of 15 820 sq. ft. Below, was a good basement.

Here, again, it was a case of constructing a new building in another and more convenient location. The new building is exactly similar in construction to the old building, except that one wing was omitted, thereby reducing the floor space to 10 540 sq. ft. In lieu of building the third wing, the City did certain additional work at the new power plant, which the State desired, and other than that which the City would have had to do simply to supply the State with a new power plant equal to the old one.

All the laundry equipment was moved from the old building to the new, the larger fixed pieces consisting mainly of four washing machines, three wringers, large dryer, starch machine, water heater, dry tumbler, large steam

mangle, 26 ironing boards, and 21 electric irons and stands. Including those which were portable, a total of 139 pieces of equipment were moved. Further, all the radiators and steam piping, electric and plumbing fixtures, were moved to and installed in the new building. At no time was laundry work entirely shut down, partial operation having been maintained either in the old building or in the new.

After the new laundry was in service the State took down the old building and salvaged much of the material, with which it constructed another building, somewhat smaller, however, than the old laundry.

Sewage-Treatment Works.—Prior to the construction of the reservoir the sewage from the Girls Industrial School flowed by gravity to and through a sewage treatment works, consisting of a screen chamber, a flat-bottom, rectangular septic tank, trickling filters, and intermittent sand filters, the effluent being discharged directly into the river. These works were at such an elevation as to be flooded by the reservoir, so new treatment works, at a higher elevation, had to be built to replace the old ones.

The new treatment works consist of a screen chamber, pumping station, septic tanks, sludge-drying beds, a dosing chamber, and intermittent sand filters, constructed in accordance with plans approved by the Ohio Department of Health. The effluent is discharged into the reservoir about 3 ft. below full reservoir level. The works were designed to treat the sewage from a population of 700.

The screen chamber is a small rectangular structure, 3 by 5 ft. in plan, and is fitted with a bar screen, the bars being spaced 1 in. apart in the clear. From the screen chamber the sewage flows to an automatically controlled pumping station of the pneumatic ejector type. The ejectors are in duplicate, of the Shone make, each ejector having a capacity of discharging 150 gal. at each operation, against a total head of 20 ft., and at least once a minute. The air compressors, of which there are two, are horizontal machines, 6 by 5 in., and of the double-acting, single-cylinder, crank and flywheel type, each compressor being driven by a 5-h.p., 220-volt, direct-current, 1 200 rev. per min., electric motor. The air receiver is 42 in. in diameter and 96 in. high. The equipment is housed in a small brick building, the ejectors being in the substructure.

From the pumping station the screened sewage is pumped to a battery of three hopper-bottom septic tanks. Each tank is 12 ft. square in plan and 13.5 ft. deep from the flow line to the bottom of the hopper. Sewage is admitted at one side of the tank, at the top, and is taken out at the opposite side. Scumboards are provided in front of both the inlet and outlet weirs. Running to the bottom of each tank is an 8-in., cast-iron, sludge discharge pipe so that sludge can be withdrawn without unwatering the tanks.

Septic tanks were chosen for simplicity in operation. The treatment works are in the country, at a sufficient distance from the nearest buildings of the school, as well as being screened from these buildings by a heavy thicket of large trees, so that the question of possible odors could be neglected. The tanks were planned to be operated in rotation, that is, in so far as the digestion of sludge is concerned. When the sludge storage space at the bottom of

one of the tanks is filled, the tank is cut out of commission and allowed to stand idle for a few months, until the sludge is digested. The digested sludge is then withdrawn and the tank put back in service. A second tank is then removed from service and allowed to stand idle, and, finally, the third tank, and so on in rotation. In other words, two tanks are sufficient for sedimentation and sludge accumulation, the third tank serving as a sludge digestion tank.

From the septic tanks sludge is first drawn into a sludge manhole and then discharged from it to the sludge-drying beds. These are three in number, two are 30 ft. square and the third, 10 ft. wide by 30 ft. long. The total sludge-drying area is 2 100 sq. ft. The beds are constructed of 9 in. of sand, supported by screened gravel, and are underdrained with 6-in. tile pipes. The sludge is confined to the beds by pre-cast concrete boards and posts which form the sides and partitions of the beds.

From the septic tanks the settled effluent flows to and through a dosing chamber and thence to the intermittent sand filters. The dosing chamber is 12 ft. wide by 18 ft. long and 3.5 ft. deep, is covered, and is fitted with four automatic timed siphons for dosing the filters, each siphon being piped so as to discharge on to one of two filters.

The sand filters, of which there are eight, are each 69 ft. square, the total sand area being 38 088 sq. ft. The banks surrounding the filter area and subdividing it into the eight beds are of earth construction, seeded and sodded. The filter sand is 30 in. in depth, the effective size being between 0.3 and 0.45 mm. and the uniformity coefficient not exceeding 2, and is supported on a bed of clean gravel in which 6-in. tile under-drains are laid.

After the new treatment works were in operation, the old works were abandoned and material from them was used by the State in miscellaneous construction on the grounds of the institution.

Acquisition of Land

There were three distinct steps in acquiring the land for the O'Shaughnessy Reservoir. They were, first, the determination of the amount of land to be taken from every riparian property affected by the reservoir project; second, the securing of a fair appraisement of the land and improvements to be taken in fee, of flood rights, and of the damage to owners resulting therefrom; and, third, the purchase of the various tracts by negotiation or condemnation, the appraisement being used as a guide for negotiation.

The elevation of the crest of the spillway, plus the maximum depth of water which it was assumed might pass over the same, plus a reasonable additional unflooded strip of land for protection and control fixed, in a general way, the elevation of the taking lines on riparian properties. The city purchased the records of the Franklin County Conservancy District, which consisted of all the data collected in connection with the study of flood relief for the Scioto River Valley, following the great flood in March, 1913, and the topographic maps of that section of the Valley embracing the proposed reservoir site were used in a tentative determination of taking lines on the various properties affected.

Abstracts of the titles of all riparian properties were secured and surveys made of the properties and of the tentative taking lines. The final taking lines were then established, taking into account the relocation of roads on the east side of the reservoir and also the requests of various land owners when the interests of the City were not prejudiced thereby. Complete property maps were then made, which showed for each property the total acreage, the acreage to be taken, and the location of the improvements on the property. These maps were turned over to the City Attorney to whom the purchase of the property had been delegated.

The first step taken by the City Attorney was the selection of a competent Board of Appraisers. The City of Columbus is in Franklin County and the O'Shaughnessy Dam and Reservoir in Delaware County. Five appraisers were selected, four from Delaware County and one from Franklin County. The importance of having a competent appraisement made is reflected in the fact that in the purchase of the 56 tracts acquired by the City, 55 were acquired by negotiation at close to the appraised value, the other being taken to Court where a jury fixed the price within 1% of the appraisement.

The appointment of the appraisers and the fixing of their compensation was authorized by an ordinance of the City Council passed January 9, 1922. The first tract of land was purchased on May 15, 1922, and the last tract on September 23, 1925. The maximum price paid for land was $175, and the minimum, $60 per acre. The land purchases were made by Messrs. Charles A. Leach, City Attorney, John L. Davies, First Assistant, and E. W. McCormick, Second Assistant. A summary of the cost of the reservoir lands, subdivided under various headings, is given in Table 3.

TABLE 3.—Cost of Reservoir Lands.

Items.	Unit.	Number of units.	Total amount paid.	Average price paid per unit.	Average price paid per acre, based on 1 155.9 acres.
Land	Acres	1 155.9	$155 026.75	$ 134.12	$134.12
Flood rights	"	51.7	1 797.00	34.72	1.55
Dwellings	Each	17	59 500.00	3 500.00	51.50
Barns	"	17	16 005.00	941.47	13.85
Other buildings	"	44	8 599.50	195.44	7.45
Springs or wells	"	28	12 050.00	430.36	10.40
Standing timber	.....	.....	5 064.00	.....	4.38
Damage to residue	.....	.....	18 580.00	.....	16.05
Other considerations	.....	.....	5 022.29	.....	4.36
Total to owners	.....	.....	$281 644.54	.....	$243.66
Fees, taxes, assessments etc	.....	.....	8 488.56	.....	7.34
Total cost	.....	.....	$290 130.10	.....	$251.00

Cost of Work

The total cost of the O'Shaughnessy Dam and Reservoir, exclusive of engineering, was $2 220 080, and a distribution of this cost, subdivided under a number of headings, is given in Table 4. No attempt has been made to dis-

tribute the cost of engineering over the several parts of the work, as it is believed that the costs as presented are of greater value than if they included a percentage for engineering, which, at best, would have to be only approximate.

TABLE 4.—COST OF O'SHAUGHNESSY DAM AND RESERVOIR.

Land, 1 156 acres			$ 290 130
Dam:			
East approach, 483 lin. ft		$ 29 790	
West approach, 403 lin. ft		13 580	
Spillway, 879 lin. ft		635 930	
East bridge abutment, 63.2 lin. ft		18 440	
West bridge abutment, 63.2 lin. ft		15 670	
Bridge over spillway, 879 lin. ft		131 530	
Pavilion:			
Substructure	$ 4 920		
Superstructure	7 940	12 860	
Outlet Gate-House:			
Substructure	$ 64 130		
Superstructure	12 800		
Pipe, gates, and equipment	24 020	100 950	
Power Gate-House:			
Substructure	$ 21 500		
Superstructure	11 470	32 970	
Approach channel to power gate-house		2 700	
Head-race below power gate-house		9 690	
Spillway channels		107 030	
Grouting		15 880	
Engineers' office		9 470	
Unclassified		10 480	1 146 970
Reservoir:			
Clearing and grubbing, 741 acres		$136 320	
Relocation of roads, 7.5 miles		233 050	
Reconstruction of Bridges:			
Rathbone Bridge, 1 046 lin. ft	$100 110		
Eversole Run Bridge, 40 lin. ft	5 180	105 290	
Moving church and cemetery		11 130	
Reconstruction at Girls Industrial School:			
Power house	$ 83 850		
Water supply	28 810		
Laundry	40 110		
Sewage treatment works	50 180		
Conduits, sewers, and miscellaneous	89 930	292 880	
Unclassified		4 310	782 980
Total cost, exclusive of engineering			$2 220 080

On the basis of a total available storage capacity of 5 341 000 000 gal., the dam and reservoir, including the land, cost $416 per 1 000 000 gal. available capacity. In Table 5, the cost is subdivided under the headings of land, dam, and reservoir; similar costs are also given, based on the total capacity of the reservoir. The difference in cost per 1 000 000 gal., between that based on available capacity and on total capacity, is just about 1 per cent.

Due to the fact that other engineering work was being designed and constructed under the supervision of the engineering organization that had charge of the design and construction of the dam and reservoir, and at the same time, it would be exceedingly difficult to distribute accurately the cost of engineering to the dam and reservoir and to this other work, notwithstanding that each assistant was required to keep a record of his time. The

difficulty is in distributing accurately the cost of supplies and expenses, and also that part of the pay-roll covering general charge and supervision of the work.

TABLE 5.—COST PER 1 000 000 GALLONS OF O'SHAUGHNESSY DAM AND RESERVOIR.

Items.	Total cost.	COST PER 1 000 000 GALLONS.	
		Total capacity, 5 400 000 000 gal.	Available capacity, 5 341 000 000 gal.
Land	$ 290 130	$ 54	$ 54
Dam	1 146 970	212	215
Reservoir	782 980	145	147
Total, exclusive of engineering	$2 220 080	$411	$416

It is possible, however, to give the total cost of engineering on the dam and reservoir and the other work referred to, which consisted mainly of the enlargement of the water-purification works and the laying of additional cast-iron pipe lines; and it may be said that all this other work would be classed under the general head of water-works construction. The total cost of engineering was $324 810, and, based on a total cost of construction of $3 217 440, the percentage for engineering amounted to 10.10. A distribution of the cost of engineering under the headings of borings and sub-surface investigations, pay-roll, supplies, and expenses is given in Table 6.

TABLE 6.—COST OF ENGINEERING.

Items.	Cost.	Percentage, based on total cost of construction of $3 217 440.
Construction:		
O'Shaughnessy Dam and Reservoir	$2 220 080	69.00
Enlargement of water-purification works	397 140	12.34
Cast-iron force mains	560 920	17.44
Cast-iron distribution mains	34 050	1.06
Miscellaneous	5 250	0.16
Total cost of construction	$3 217 440	100.00
Engineering:		
Borings and sub-surface investigations	$ 10 800	0.34
Pay-roll	271 280	8.43
Supplies	11 110	0.35
Expenses	31 620	0.98
Total cost of engineering	$ 324 810	10.10
Summary:		
Construction	$3 217 440	100.00
Engineering	324 810	10.10
Total cost	$3 542 250	110.10

Beginning and Completion of Work

As stated at the beginning of this paper, the construction of an additional dam and reservoir to provide enlarged storage of raw water was recommended by one of the writers in a report dated August 2, 1919. At a general election held on November 4, 1919, the issuance of bonds in the sum of $3 000 000 for this improvement, together with the extension and enlargement of other parts of the water-supply system, was approved by popular vote.

On March 15, 1920, the City Council passed an ordinance which provided for the issue and sale of these bonds, and on April 12, 1920, an ordinance was passed which provided for the establishment of the Bureau of Water-Works Extension in the Division of Water for the carrying out of these improvements.

The selection of the site for the new dam was made after twenty-six prospective sites between the Village of Dublin and the Powell Road Bridge had been studied. The contract for "The O'Shaughnessy Dam and Appurtenant Works" was awarded on June 6, 1922, to the Thompson-Starrett Company, of New York, N. Y., at an estimated cost of $1 165 511. About two weeks after the signing of the contract, the clearing and grubbing of the site and the erection of the contractor's plant were started. The first concrete was poured on November 1, 1922, and the dam and bridge were completed and ready for use on June 1, 1925.

Construction of the reservoir was carried on coincident with the construction of the dam, and all the work was practically completed by October 1, 1925.

Summary of Principal Data Relating to the O'Shaughnessy Dam and Reservoir

For convenience of reference the principal data relating to the dam and reservoir may be summarized, as follows:

Date of construction	1922-25
Total length of dam, in feet	1 750
Length of masonry part of dam, in feet	879
Length of earth approaches, including concrete abutments, in feet	871
Height of spillway above low water in river, in feet	64
Height of dam from rock foundation to crest of spillway, in feet	84
Height of dam from rock foundation to roadway level, in feet	105.25
Height of roadway above spillway, in feet	21.25
Maximum width of dam at base, in feet	110
Width of paved roadway, in feet	17.5
Width of sidewalks, in feet	4
Net length of spillway, in feet	765.7
Maximum flood capacity of spillway, in cubic feet per second	110 000
Depth of water over spillway with maximum flood, in feet	12.5
Total available storage capacity, reservoir full, in gallons	5 341 000 000

Average depth of water, reservoir full, in feet	19.8
Maximum depth of water, at dam, reservoir full, in feet	69
Length of reservoir, in miles	8
Average width of water surface, reservoir full, in feet	855
Area of water surface, reservoir full, in acres	829
Area of land cleared and grubbed, in acres	741
Area of land purchased, in acres	1 156
Total cost of dam and reservoir, exclusive of engineering	$2 220 080

Contractors

The dam and reservoir were constructed mainly by contract work, the principal contracts and names of the contractors being as follows:

O'Shaughnessy Dam	Thompson-Starrett Company
Diamond drill core borings	Giles Drilling Company
Reconstruction at Girls Industrial School:	
Replacement of buildings	Middle States Construction Company
Moving and replacing equipment and miscellaneous structures	Pitt Construction Company
Replacement of boilers	Heine Boiler Company
Replacement of pumping equipment	Factory Engineering and Sales Company
Centrifugal pumps	Elcon Engineering Company
Relocation of Roads:	
O'Shaughnessy Dam to Rathbone	Uncapher and Gillespie
Rathbone to Bellepoint	J. S. McCann
Reconstruction of Bridges:	
Rathbone Bridge	E. M. Scully
Eversole Run Bridge	Uncapher, Wooley, and Gillespie
Clearing and Grubbing:	
Northern and southeast portions	W. D. Winchell and Company
Southwest portion	J. Livingston and Company

In the construction of the dam, the Thompson-Starrett Company was represented by N. M. Loney, M. Am. Soc. C. E., Vice-President of the Company, and in the moving and replacing of equipment and miscellaneous structures at the Girls Industrial School, the Pitt Construction Company was represented by E. D. Harshbarger, Assoc. M. Am. Soc. C. E., President. In the clearing and grubbing, J. Livingston and Company was represented by L. I. Fletcher, Affiliate, Am. Soc. C. E., Resident Manager.

Legislative Procedure

In making the improvements described in this paper it was necessary to comply with State Statutes, with the City Charter, and with City Ordinances, all of which specify terms and conditions governing the expenditure of public

funds. The Legislative Acts required to legalize the carrying out of the work and to both provide and disburse the necessary funds were, as follows:

Statute by State Legislature:		
To authorize agreement with the City to flood State property	1	1
Vote of Citizens at General Election:		
To approve the issue and sale of bonds	1	1
Ordinances by City Council:		
To authorize the issue and sale of bonds	6	
To provide for the engineering organization	9	
To provide for the purchase of supplies and equipment	5	
To authorize contract work	29	
To authorize bed-rock surveys	4	
To authorize the acquirement of property	41	
To authorize miscellaneous projects	22	116
Resolutions by City Council:		
To name the dam	1	
To submit plans to the State Department of Health	2	
To advertise contract work	1	
To appropriate property by condemnation	3	
To modify contracts	5	12
Total Legislative Acts		130

Effect of Additional Storage on Softening of the Water

The O'Shaughnessy Dam was completed June 1, 1925. From May 1 to October 31, 1925, the average daily flow of the Scioto River was 36 000 000 gal. and the average daily water demand of the city 27 500 000 gal. The flow of the river during 1925 was below normal and would have been a good year for showing the effect of storage in reducing the hardness of the raw water. However, the dam was not completed in time to permit the storage of the soft flood waters of the late winter and early spring months. During 1926 the average daily flow of the river from May 1 to October 31 was 630 000 000 gal., and the average daily demand of the city for the same period was 29 000 000 gal. Consequently, 1926 was not a good year to show the effect of storage, but it is the only year for which data are available.

At the upper end of the O'Shaughnessy Reservoir the influent is made up of the flow from Mill Creek, a tributary having a water-shed of 186 sq. miles, and from the Upper Scioto, which has a water-shed of 770 sq. miles above the confluence with Mill Creek. The water-shed tributary to the reservoir is, therefore, 956 sq. miles; and for further reference it should be stated that the Scioto River has a water-shed of 988 sq. miles at the O'Shaughnessy Dam, 1 052 sq. miles at the Julian Griggs Dam, and 1 076 sq. miles at the water purification works.

Daily samples of water, collected from Mill Creek and the Upper Scioto, were analyzed for mineral content, and the average values weighted on the basis of the contributing water-sheds. The average total hardness of the raw-

water inflow into the O'Shaughnessy Reservoir, from May 1 to September 10, 1926, was 313 parts per million, and the average hardness at the intake to the water purification works, about 22 miles down stream, and with the two reservoirs intervening, which were full all the time and have a total combined storage capacity of 7 120 000 000 gal., was 278 parts per million, a reduction of 35 parts per million in total hardness. During the same period the raw-water draft for the City was 3 911 000 000 gal., and for 1926 the average cost of chemicals for removing 1 part of hardness from 1 000 000 gal. of raw water was \$0.104. The apparent saving in the cost of chemicals used in water-softening resulting from storage during the period mentioned, therefore, may be said to be $3\,911 \times 35 \times \$0.104 = \$14\,200$. Additional data, which are being collected this year (1927), should throw more light on the subject.

Engineering Organization

The engineering work was carried out by the Bureau of Water-Works Extension, a temporary bureau created in the Division of Water and Sewage Disposal by ordinance of the City Council. The bureau began to function on May 1, 1920. Of the three writers, John H. Gregory, M. Am. Soc. C. E., was Consulting Engineer, and C. B. Hoover, M. Am. Soc. C. E., Engineer in Charge, of the Bureau; C. B. Cornell, M. Am. Soc. C. E., was Construction Engineer in charge of the work at the dam and reservoir. Following the death of Mr. Jerry O'Shaughnessy, Superintendent of the Division of Water and Sewage Disposal, Mr. Hoover was appointed Superintendent of the Division on December 1, 1921, and, in addition to his duties as Superintendent, continued to serve as Engineer in Charge of the Bureau of Water-Works Extension until October 1, 1925, when the work of the Bureau was practically completed. For the first two years, A. R. Holbrook, Assoc. M. Am. Soc. C. E., was Office Engineer, and following his resignation he was succeeded by John C. Prior, M. Am. Soc. C. E. The reservoir surveys were in charge of V. C. Norton, Field Engineer. The Hon. James J. Thomas was Mayor of Columbus and W. H. Duffy, Director of Public Service.

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PAPERS AND DISCUSSIONS

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THE TREATMENT OF THE WATER SUPPLY OF THE CITY OF COLUMBUS, OHIO*

By Charles P. Hoover,† Esq.

Synopsis

A description of construction features of the Water Softening and Purification Works as originally built at Columbus, Ohio, will not be given in this paper. This has been done‡ by John H. Gregory, M. Am. Soc. C. E., who was Engineer in charge of the design of the plant. Operation and new features only will be discussed.

Original Water Supply

Prior to the time the Purification Plant was built, the water supply of Columbus was not safe for drinking. It was badly polluted with sewage, always turbid, and, following heavy rains, extremely muddy. The turbidity of the Scioto River averages 75 parts per million. During floods this rises to 2 250 parts per million, meaning, that a day's supply carries from 200 to 300 tons of mud in suspension.

The bacterial count (37° cent., 24 hours) averages 5 665 per cu. cm. The *B. Coli* count sometimes reaches 100 000 per 100 cu. cm. In 1908, the typhoid fever death rate reached 138 per 100 000 population. During the summer of that year there were 1 500 cases of typhoid.

The original supply of water was unsatisfactory for domestic or industrial purposes because it was highly mineralized. The average hardness of the present raw water from 1909 to 1926, inclusive, was 272 parts per million, with a maximum of 495 parts per million. This hardness is due to calcium carbonate, magnesium carbonate, calcium sulfate and magnesium sulfate.

Note.—Written discussion on this paper will be closed in **August, 1928.**

* Presented at the Fall Meeting of the Society at Columbus, Ohio, October 12, 1927.

† Chemist in Chg., Water Softening and Purification Works, Columbus, Ohio.

‡ "The Improved Water and Sewage Works of Columbus, Ohio," by John H. Gregory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXVII (1910), p. 206.

There are 68 570 lb., or more than a carload, of these compounds in a day's supply. There is enough calcium to make 48 tons of hydrated lime and the magnesium in one day's supply would make 62 500-qt. bottles of milk of magnesia.

Calcium and magnesium salts are very objectionable. They form deposits in boiler tubes and hot-water heating systems, resulting in large heat losses. They also re-act with soap to form slimy, insoluble compounds, which can be seen in the water before lather begins to form. Soap has no cleansing action until it lathers, therefore, much of it is wasted in hard water.

Before the water supply of the city was softened, cistern water was used almost generally for domestic purposes. More than 17 000 cisterns were in use in Columbus, when the municipal plant was built. Cisterns are only makeshifts for a suitable municipal water supply. The water in them is hardly ever adequate, and is almost always dirty and, sometimes, not even soft. They have now been practically abandoned in Columbus.

Water from individual private wells was used for drinking purposes, but many of these wells were contaminated with sewage. They were condemned by the Board of Health after the Purification Plant was put into service.

Status of Water Filtration and Water Softening in 1904

At the time (1904) the Columbus plant was being designed, only two large mechanical water filtration plants of concrete construction were in operation, one at Little Falls, and one at New Milford, N. J. Valuable and helpful experimental work on water purification had been done by George W. Fuller, M. Am. Soc. C. E., who was one of the consulting engineers on the design of the plant, at Louisville, Ky., and Cincinnati, Ohio, on the treatment and filtration of water,* but no large combined water-softening and filtration plant had been built. There was little or no precedent to follow. Under the supervision of George A. Johnson, M. Am. Soc. C. E., bottle and barrel water-softening experiments were made at the Columbus Sewage Testing Station. The results of these experiments were used as a guide in the design of the plant. The demand for the new plant was so urgent that there was no time to investigate the Columbus problem extensively, as was done at Louisville and Cincinnati.

Flexible Plant Built

The plant, as built, was provided with facilities for softening and filtration. The design was so elastic that the operating process could be modified without making elaborate changes. Numerous places for applying the chemicals were provided. The water could be separated and treated in portions, also the time of mixing could be varied. The mixing and settling basins could be operated in series, or parallel, or in series parallel. Raw water could be applied at many points to correct for over-treatment.

As a result of this flexibility, after twenty years of continuous use, the plant is being operated as effectively as the most recently built plants. No extensive or costly additions and changes have been required to accomplish this; it has

* Report of Water Purification at Louisville, Ky.; and Report of Water Purification at Cincinnati, Ohio.

never been necessary to shut down for repairs, changes, or enlargements. The plant has been shut down completely only once since it was put in operation in 1908. This was necessary during the 1913 flood, when one of the force mains, laid in the river bed, was washed out, making it impossible to maintain pressure in the distributing system and this shut down lasted for only eight hours.

Character of Raw Water Changes Suddenly

The design of the plant was also complicated because the Scioto River, from which Columbus takes its supply, is very flashy. It is sometimes clear, sometimes muddy, sometimes soft, and sometimes hard; and as the waterworks is not provided with large settling basins, in which to store raw water to equalize these fluctuations, the plant must take whatever is flowing in the river. Table 1 shows the fluctuations in the raw-water analysis during a 48-hour period from March 21 to 23, 1916.

TABLE 1.—Character of Scioto River Water.

Date.	Turbidity.	Carbonate hardness.	Non-carbonate hardness.	Total hardness.
March 21.....	22	189	96	285
March 22.....	678	133	63	196
March 23.....	2 250	46	18	64

It was known by those who designed the plant that these sudden changes in the analysis of the raw water would occur. Specially designed and readily adjustable devices, capable of feeding small and large quantities of chemicals, were provided.

These sudden changes require constant supervision to treat the water properly. A chemist is kept on duty day and night making tests once every hour. At plants where only coagulation and filtration are practiced a change in the character of the raw water coming to the plant does not present as serious a problem to the operator as it does at a water-softening plant. In the former it is only necessary to increase the application of the coagulating re-agent; whereas, at a water-softening plant, it is not only necessary to increase the application of the coagulating re-agent but also to diminish the quantity of softening chemicals.

The addition of too much lime gives the water a caustic reaction. The addition of too small a quantity of lime may prevent the coagulant from completely precipitating, resulting in the passage of alum floc through the filters and the production of a turbid effluent. Lime, unless added in exactly the right quantity, may be detrimental to successful purification and clarification processes.

Process Used in Treatment of Water Supply

The water flows, from the river (Fig. 1), through screens into a suction well. From the well it is pumped through a Venturi meter into the receiving

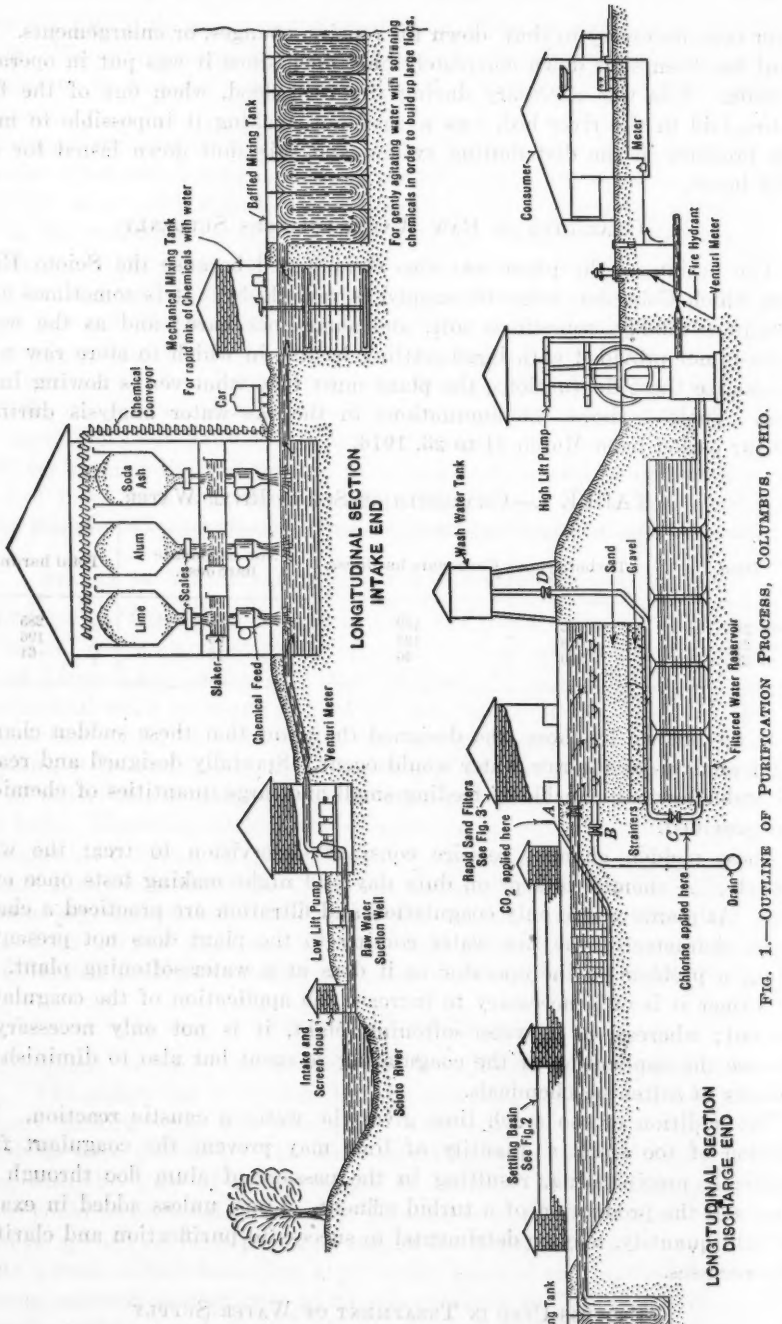

FIG. 1.—OUTLINE OF PURIFICATION PROCESS, COLUMBUS, OHIO.

compartment at the Purification Works. Here, it is divided into approximately two equal parts. One part without treatment flows directly to the baffled mixing tanks and the other part, to which is added the entire quantity of lime, soda-ash, and alum solutions necessary to soften and clarify the entire supply, flows to mechanical mixing tanks.

The portion of water receiving the chemical charge is mixed for about 10 min. in square tanks provided with rotating paddles. During this time chemical reactions between the chemicals added to the water and the mineral salts already present in the water are effected. This reaction results in the formation of an extremely heavy gelatinous precipitate, which is largely due to the mass action effect of over-treatment. Under these circumstances the magnesium compounds present in the water are readily precipitated. Lime combines with magnesium to form magnesium hydroxide, a bulky gelatinous precipitate, which coagulates as readily as the precipitates formed by sulfates of alum or iron. It is, however, extremely difficult to precipitate magnesium unless excess lime is applied. This is the reason why separate treatment is used.

This over-treated portion of the water, with floc already formed, joins the other portion of untreated raw water at the entrance of the baffled mixing tanks, and the two are mixed together for about an hour. Floc formation having already been started, further floc-forming reactions take place readily in the baffled mixing tanks. The size of the floc grows as the water rolls over and under the baffles.

From the baffled mixing tanks the water flows to the settling basins (Fig. 2). They are six in number. The mud and precipitated chemicals settle to the bottom of these basins, practically all of it being removed in the first two. It is necessary to wash these basins every three or four weeks. The last four basins are washed only once or twice a year.

Just before the clear softened water leaves the settling basins, carbon-dioxide gas is applied in order to neutralize any excess lime. It also converts the normal carbonates to bi-carbonate form. This reaction stabilizes the water, and prevents after-reactions, or deposits, in the filters or distributing system.

After carbonation, the water discharges from the settling basins into the filters. Here, it is filtered through 24 in. of sand and 18 in. of gravel. As it discharges from the filters into the filtered water reservoir a small quantity of chlorine gas is added, the quantity averaging only 0.15 parts per million. The water meets the U. S. Treasury standards for bacterial purity without the addition of chlorine, but the small quantity mentioned is added as an additional safeguard. From the filtered reservoir the water is pumped by means of high-lift pumps (Fig. 3) into the distributing system to the consumers.

Advantages of Lime Treatment

Lime is added to water primarily to soften it, but it has been discovered that, when added in sufficient quantity to produce residual or excess lime (caustic alkalinity), it kills intestinal and pathogenic bacteria, rendering the water safe bacterially.

The lime-softening process also increases the efficiency of sedimentation because better coagulation is effected. The bulk of the precipitate formed by the softening reaction is about twenty times as great as could be obtained by ordinary coagulation processes. This large, bulky precipitate of calcium carbonate and magnesium hydroxide entangles fine particles of suspended matter, bacteria, and other impurities in the water, and settles very rapidly.

Lime-softened and settled water is low in turbidity, color, and organic matter, these having been absorbed or dragged down mechanically by the precipitates. In addition, the water will be free from objectionable gases, such as carbon dioxide and hydrogen sulfide, these having been neutralized. The riddance from bacteria, turbidity, iron, color, organic matter, and objectionable gases by lime treatment, lessens the cost of filtration because filter runs are long, and filtration areas may be reduced.

Lime softening is well adapted to the treatment of a polluted hard water, such as is treated at Columbus. It is possible during flood times, when tastes and odors are feared, to produce a satisfactory potable water without having to depend on chlorine to kill bacteria.

Application of Chemicals to Water

The lime, soda ash, and alum are now applied to the water at practically one point. When the plant was first put in operation the lime was applied to approximately 25% of the raw water, which was diverted through saturators in order to make a saturated lime water containing 60 grains of calcium oxide per gal. This saturated lime water was then mixed with 50% more of the raw water, making a total of 75% lime-treated water. The soda ash was mixed with the remaining 25% of the raw water for about 20 min. and then mixed with the lime-treated water. This was given up because it was found to be extremely difficult to produce a uniform saturated lime solution.

It was also discovered that considerable lime was lost in the sludge, which deposited in the bottom of the saturators. There was no noticeable advantage in treating the water separately with lime and soda ash and then mixing the two waters together. The resultant reaction is the same as when they are added together. The process was simplified by applying the chemicals at one point, but the split treatment is still used because the hardness of a given quantity of raw water with a given quantity of chemicals will be reduced more than when added in the usual way.

Water not "Doped" with Chemicals

There is a general belief that the use of chemicals for softening water is not desirable. There is usually prejudice against it. Wherever water-softening plants have been built, this prejudice has been almost entirely overcome. The citizens of municipalities operating a water-softening plant, especially in those cities where water softening has been practiced for some years, now understand that the water is not "doped". They have learned there is less mineral matter or chemicals in the softened water than in the raw water. The chemicals, which are added in order to soften and purify the water, combine with the soluble mineral salts already present in solution, and new

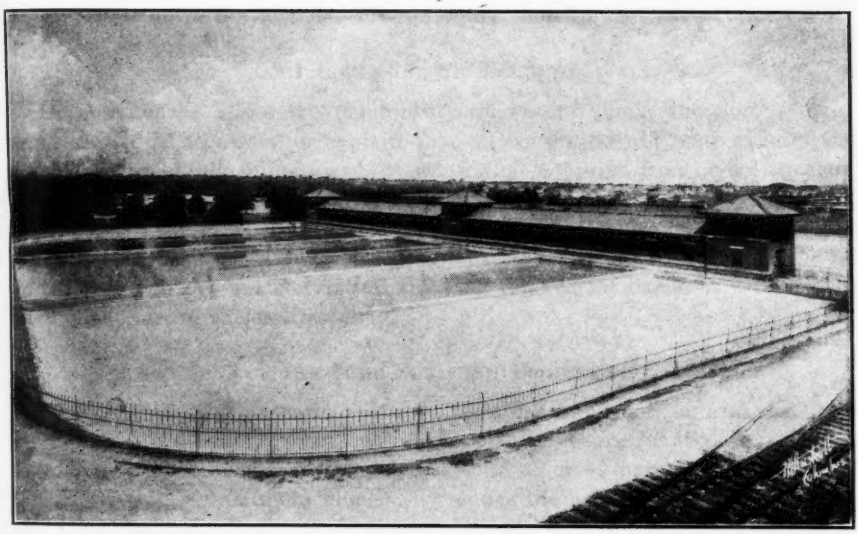

FIG. 2.—VIEW OF SETTLING BASINS, COLUMBUS, OHIO, PURIFICATION WORKS.

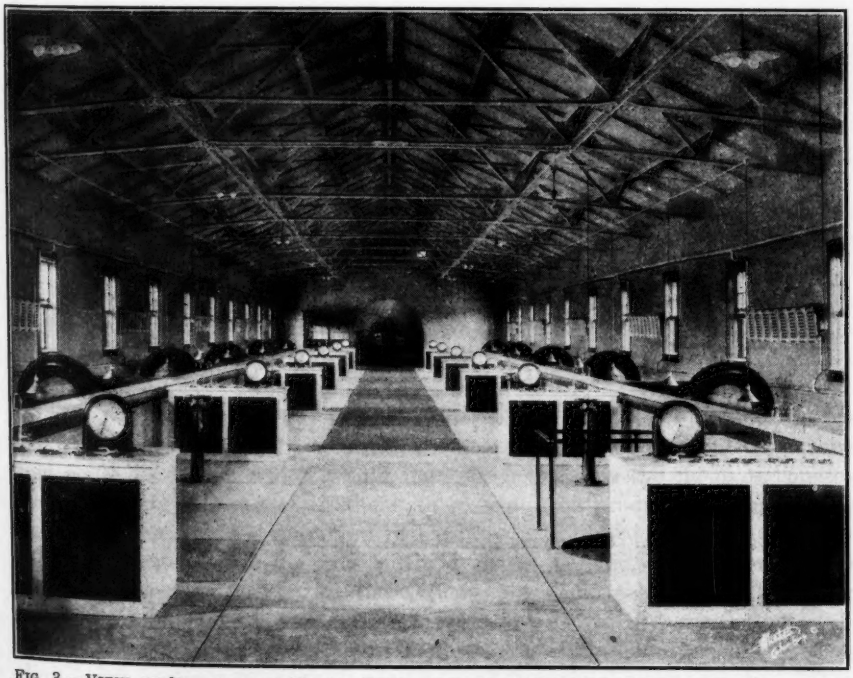

FIG. 3.—VIEW OF INTERIOR OF FILTER ROOM, SHOWING HIGH-LIFT PUMPS, COLUMBUS, OHIO, PURIFICATION PLANT.

insoluble compounds are formed. These are easily removed, leaving less mineral content in the softened filtered water than in the raw water.

Effect of Softening on Taste of Water

Does softening affect the palatability of water? This question is often asked and it is necessary to explain that all waters do not taste exactly the same. Water highly mineralized, or water that contains iron or sulfur compounds, has a more pronounced taste than water free from these compounds, but there should be no difference in the taste of softened carbonated water and that of a natural water of the same hardness. Also, there seems to be no evidence to support the contention that the removal of hardness salts makes water either more or less healthful.

Limitations of the Lime Soda-Ash Process of Softening

The product of the reaction between the lime and soda ash, and the hardness salts in the water, is not entirely insoluble. Consequently, it is impossible to reduce the hardness to much less than 35 or 40 parts per million with lime and soda-ash treatment alone. To reduce the hardness to this low figure an excess quantity of chemicals must be added.

It has been the experience at Columbus that when the hardness has been reduced to about 85 parts per million, the cost of further removal of hardness is proportionately higher. This is due to the difficulty of precipitating magnesium compounds.

Methods for further reducing hardness by high temperature, excess chemical treatment, split treatment, addition of aluminum salts and excess lime neutralized by carbonation have been described elsewhere by the writer.*

Lime Scale in Pipes and Filters

Lime-softened water is a saturated solution of calcium carbonate, and like any saturated solution is, of course, not stable. Difficulty has resulted from the deposition of calcium and magnesium scale in the filter sand at the plant and in the pipe system of the city. It has long been known that carbon dioxide applied to lime-softened water would stabilize it, because it re-acts with the slightly soluble non-carbonates and changes them to high soluble bi-carbonates, which are not easily precipitated.

For some time doubt was felt as to whether or not carbon dioxide was the best re-agent for corrective treatment. It was realized that if an inexpensive acid could be found, which would not only neutralize the mono-carbonates, but precipitate all or part of them, there would be produced a stable water and at the same time a reduction in hardness.

Experiments were made with sulfuric, silicic, boracic, oxalic, tartaric, and phosphoric acids. Among all these, phosphoric acid seemed best, but, was given up because of its cost and an after reaction, which developed when the water was heated. Many different methods may be used to produce carbon-dioxide gas, and several methods may be used to apply the gas to water. These

* *The American City*, Vol. XXXVII, September, 1927.

TABLE 2.—COMPARATIVE AND AVERAGE RESULTS OF OPERATION FROM 1909 TO 1926, BOTH INCLUSIVE.

Year.	Volume of water, in million gallons per 24 hours.	Total Hardness, in Parts per Million.			Carbonate Hardness, in Parts per Million.			Non-Carbonate Hardness, in Parts per Million.			Magnesium, in Parts per Million.		
		River water.	Filtered water.	Percentage removed.	River water.	Filtered water.	Percentage removed.	River water.	Filtered water.	Percentage removed.	River water.	Filtered water.	Percentage removed.
1909	14.8	253	98	63	150	40	73	102	53	48	21	10	53
1910	15.5	270	85	68	158	50	68	113	35	60	23	11	52
1911	15.6	245	84	61	150	45	71	95	38	60	21	10	51
1912	17.5	232	80	67	146	46	69	75	33	56	18	10	47
1918	18.3	271	88	67	160	61	62	111	28	75	23	12	46
1914	18.4	297	79	73	166	47	72	131	32	76	27	10	63
1915	17.7	279	88	69	170	51	70	108	36	67	26	9	59
1916	19.8	270	110	61	176	55	69	108	58	44	25	13	48
1917	21.6	278	125	55	162	45	70	126	81	36	22	12	46
1918	23.8	306	125	59	170	59	65	135	66	51	26	13	50
1919	22.7	278	106	62	160	51	68	113	55	51	22	11	50
1920	23.6	266	109	59	167	53	68	97	56	42	20	10	50
1921	21.4	269	100	63	162	54	67	107	46	57	24	11	54
1922	22.0	278	101	64	166	59	65	112	42	58	25	11	56
1923	23.9	265	95	64	156	54	66	109	41	63	22	10	55
1924	24.0	299	99	67	175	62	65	121	37	70	27	10	63
1925	26.4	298	92	69	161	59	63	137	32	77	27	9	67
1926	27.5	246	90	63	146	50	66	100	39	61	20	8	60
Average....	20.8	272	97	64	161	52	68	111	45	59	23	11	52

TABLE 3.—QUANTITIES AND COST OF CHEMICALS.

Year.	Lime.		Soda-Ash.		Alum.		Bleach and Liquid Chlorine.	
	Tons.	Cost per ton.	Tons.	Cost per ton.	Tons.	Cost per ton.	Tons.	Cost per ton.
1909	2 467	$ 5.75	1 402	$17.50	624	$19.00*	..	.....
1910	3 081	5.80	2 164	17.50	423	18.00*	..	.....
1911	3 660	5.42	1 776	17.50	590	17.50*	..	.....
1912	3 269	5.27	1 583	15.20	895	17.15*	22	$ 27.50‡
1913	3 629	5.17	2 895	12.88	711	17.10*	17	27.80‡
1914	4 550	5.27	3 540	13.88	860	16.75*	14	29.20‡
1915	3 970	5.17	2 383	15.14	69	16.69*	22	34.80‡
1915	.....	.....	.....	.....	805	7.27†	..	.....
1916	4 550	5.17	1 975	62.00	823	20.00†	19	85.00‡
1917	4 206	7.05	1 835	60.00	943	10.00†	24	70.00‡
1918	4 810	9.22	3 005	50.60	1 113	14.62†	25	54.56‡
1919	5 149	10.21	2 748	40.50	1 156	14.58†	16	60.00‡
1920	5 307	11.90	2 149	43.60	1 304	15.52†	23	67.72‡
1921	5 166	12.05	2 679	42.54	985	13.39†	29	68.89‡
1922	5 487	10.98	2 891	28.81	1 275	12.50†	11	63.00‡
1922	.....	.....	.....	.....	.....	.....	4	112.00§
1923	5 594	10.62	3 246	30.95	1 173	11.20†	8	116.30§
1924	7 192	7.77	3 742	29.67	1 267	11.95†	6	101.85§
1925	7 769	7.25	4 303	29.86	1 384	11.66†	5	122.50§
1926	7 688	6.37	3 063	29.75	1 696	12.80†	6	157.50§

* Crystal alum purchased in the open market.
† Cost of materials. Sulfuric acid cost $14.63 per ton, f. o. b. Columbus, Ohio. Bauxite cost $14.96 per ton, f. o. b. Columbus, Ohio, 1926.
‡ Bleach.
§ Liquid chlorine.

TABLE 4.—AVERAGE COST OF CHEMICALS.

For period.	Lime.	Soda-Ash.	Alum.
1909 to 1915, inclusive.....	$5.41	$15.66	$16.18
1916 to 1921, inclusive.....	9.27	49.87	14.68
1922 to 1926, inclusive.....	8.60	29.81	12.02

TABLE 5.—COST OF TREATMENT PER MILLION GALLONS.*

Date.	Chemicals.	Labor.	Other.	Total.	Cost for chemicals per part removed, per million gallons.
1909	$ 9.77	$	$	$	$ 0.061
1910	11.40	4.99	1.39	17.78	0.062
1911	10.75	4.31	2.23	17.29	0.067
1912	8.95	3.64	0.84	13.43	0.063
1913	10.30	3.57	1.09	14.96	0.056
1914	12.53	3.79	1.09	16.32	0.057
1915	10.50	3.95	0.89	15.34	0.055
1916	22.70	3.44	0.83	26.97	0.134
1917	19.15	3.68	1.60	24.43	0.125
1918	27.60	4.60	3.35	35.55	0.153
1919	22.10	5.31	....	27.41	0.128
1920	20.80	5.86	1.35	28.01	0.132
1921	29.19	7.09	1.35	37.63	0.173
1922	22.13	6.69	1.46	30.28	0.125
1923	20.90	6.57	0.93	28.40	0.123
1924	21.17	6.79	0.93	28.89	0.106
1925	22.00	6.78	0.84	29.62	0.107
1926	16.38	6.42	1.11	23.91	0.105
Unweighted average.....	$18.15	$5.15	$1.25	$24.54	$0.102

* Of the cost of chemicals, 88% was for lime and soda-ash.

were investigated over a period of seven years before starting the design of the carbonation plant, which has now been built.

Carbonation of a softened water makes it resemble natural water. If a sample of lime-softened water were submitted to a chemist for analysis, he could very easily tell that it was softened. The results of his analysis would show that the water was different from natural waters. If, however, a sample of lime-softened water that had been carbonated were submitted to him, he could not tell the source of the water from the results of his analysis. This water would be similar in all respects to any natural water of similar hardness.

It should be understood that the recarbonated water does not contain any free carbon dioxide, as only a sufficient amount to combine chemically with the normal carbonates is added, consequently carbonation of water as practised at Columbus should not make it corrosive.

The results of operation and quantities and cost of chemicals used, are given in Tables 2, 3, and 4. Table 5 shows the cost of treatment per 1 000 000 gal. and the cost of removing 1 part per million of hardness from 1 000 000 gal.

Fig. 4 shows the reduction in typhoid fever after filtration. There were five deaths from typhoid fever during 1926, three of which were resident and two non-resident. Assuming the population to be 285 000,* would give the city a total death rate from typhoid fever of 1.7 per 100 000, a resident death rate of 1.0 per 100 000, and a non-resident death rate of 0.7 per 100 000.

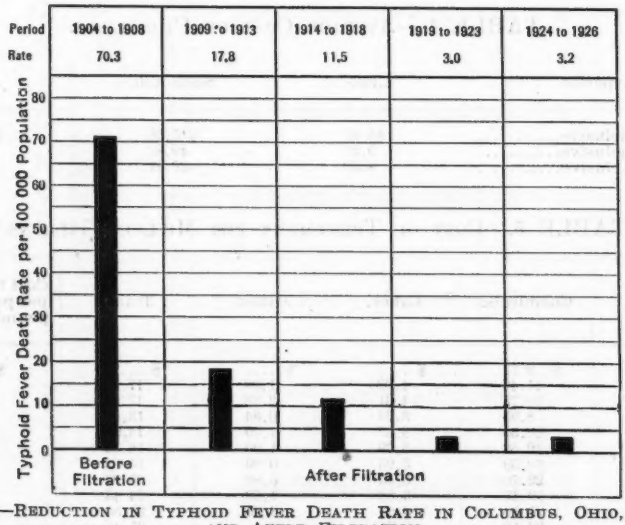

FIG. 4.—REDUCTION IN TYPHOID FEVER DEATH RATE IN COLUMBUS, OHIO, BEFORE AND AFTER FILTRATION.

IMPROVEMENTS AND ADDITIONAL FEATURES ADDED TO THE PLANT

Handling Lime.—Originally the lime was handled and stored in canvas bags. In order to increase the storage capacity, and to lessen the labor and cost of handling the lime, a concrete storage bin was built. Lime is elevated into it by a conveyor. A cross-conveyor carries it to daily supply hoppers.

* Estimate made by Ohio State Bureau of Vital Statistics.

From the daily supply hoppers the lime drops into Richardson automatic weighing scales, which automatically weigh it into solution tanks from which it is fed into the water through a chemical feed regulator.

Alum Made at Plant.—It will be noted by examining Table 3 that whereas the cost of lime and soda ash has advanced since 1915, the price of alum is lower. The reason for this is the manufacture of alum at the plant.

Bauxite and sulfuric acid are boiled in lead-lined tanks until a basic solution of aluminum sulfate is obtained. The solution is then diluted with water (usually enough water is added to make 500 gal. of the solution, equivalent to 1 ton of 17% Al_2O_3 alum), and measured as needed into alum solution tanks. There it is diluted with sufficient water to make a standard solution, which is then applied to the water under treatment. By this process five distinct steps in alum making are eliminated—filtering, concentrating, crystallizing, grinding, and re-dissolving. The plant comprising lead-lined boiling tanks, an alum-measuring tank, one acid-measuring tank, storage tanks for sulfuric acid, a pulverizer for bauxite, conveying, elevating, and transmission machinery, bauxite storage bins and weighing device, piping, valves, and fittings, is shown in Fig. 5.

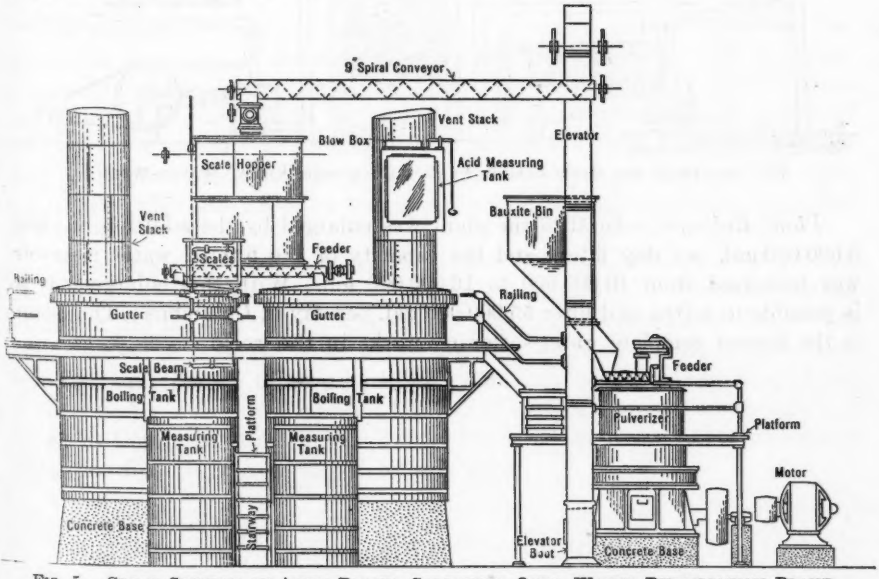

FIG. 5.—CROSS-SECTION OF ALUM PLANT, COLUMBUS, OHIO, WATER PURIFICATION PLANT.

Recarbonation Plant.—The gas-making and distributing plant consists of six principal parts (Fig. 6), a gas producer, a gas burner, a return tubular boiler, a steam-driven air compressor, a combination scrubber and drier, and a diffuser. In operation, coke is burned in the gas producer, which is a closed furnace with a controlled air supply furnished by a blower. The products of combustion pass from the gas producer to a gas burner, where they are mixed with air and burned to complete combustion under a boiler. The steam produced in this boiler is used to drive the air compressor and blower in order

to make a closed cycle for the process. The plant is located in the boiler-room of the pumping station and the steam produced is connected to the main steam line. It has been found that a small amount of steam from the main steam line is required for operating the compressor and blower. The air compressor takes its suction from the boiler stack of the producer gas-burning boiler, through a scrubber and drier, and forces the washed gases through diffusers into the water to be carbonated. Making producer gas from coke, and burning it under a boiler, is perhaps the most economical method of producing carbon-dioxide gas. The steam generated is readily available, and it is possible to produce a gas containing as much as 17 or 18% of carbon dioxide, thus requiring an air compressor of only one-fourth the capacity required if coke were burned in an open burner.

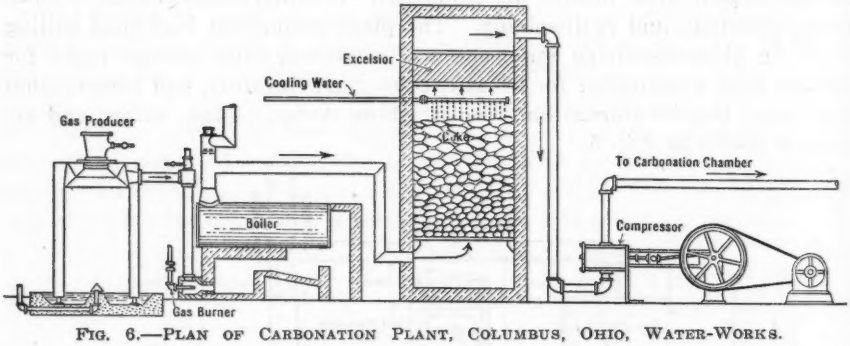

FIG. 6.—PLAN OF CARBONATION PLANT, COLUMBUS, OHIO, WATER-WORKS.

Plant Enlarged.—In 1923 the plant was enlarged by the addition of eight 3 000 000-gal. per day filters and the capacity of the filtered water reservoir was increased from 10 000 000 to 16 500 000 gal. With this enlargement it is possible to soften and filter 55 000 000 gal. per day at Columbus. This plant is the largest complete water-softening works in the world.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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CROWN STRESSES IN A SKEW ARCH

By J. Charles Rathbun,* M. Am. Soc. C. E.

Synopsis

The purpose of this investigation was to devise a method of finding the stresses at the crown of the ring of a cast skew arch model and to check the results obtained by laboratory experiments with those obtained by theoretical calculations. Unfortunately, the opportunity did not present itself to rebuild the apparatus in order to produce better results and as such an opportunity does not appear imminent, the data thus far obtained are presented for consideration. This investigation has grown out of a study made by the writer in 1923-24.†

The analysis presented at that time was of a purely theoretical nature. The writer was unable to find experiments either refuting or corroborating the work; nor did the discussions of the paper cite any experiments or theory that could be used as a check. The writer hoped to check the various deductions by a continuation of these experiments, but this has not been practicable. A study of the effect of horizontal loads is of especial interest as brought out by A. H. Beyer, M. Am. Soc. C. E.,‡ and it is hoped that further discussion along this line may be developed by this paper.

General

In order that the notation used herein and in the Appendix may be more easily understood, Fig. 1 from the 1924 paper is reproduced as the writer's Fig. 1.

In the present experiments, arches cast from plaster of Paris were secured in a framework made for the purpose (see Figs. 2 and 3). One abutment was

Note.—Written discussion on this paper will be closed in **August, 1928.**

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† "Analysis of the Stresses in the Ring of a Concrete Skew Arch," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 611.

‡ *Loc. cit.*, p. 657, Conclusion (*e*).

held as rigid as possible while the other was secured to a steel frame. To this frame were attached six dynamometers or spring balances by means of which the reactions of the free abutment could be obtained. At two positions on the arch ring were clamped steel collars. These positions were considered as the abutments of the arch ring for the purpose of computation. The dynamometers were adjusted as the load was applied, so that the two collars remained in the same position relative to each other throughout the test.

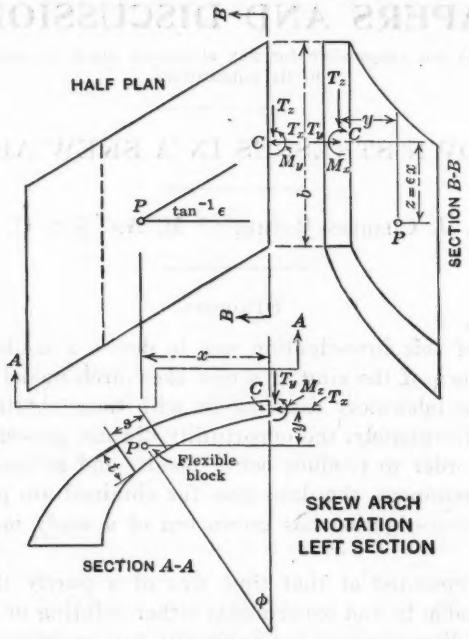

FIG. 1.

Of necessity, in considering the stresses in the ring of a skew arch the magnitude, direction, and location of forces in space must be considered, in contradistinction to forces in a definite plane, as in the right arch theory.

To define the resultant of a system of forces completely requires only six quantities. These may be taken as the components parallel to any three assumed rectangular axes and the moments about these axes. As all these quantities are mutually independent, the six are necessary as well as sufficient, and it is, therefore, important to have six and only six independent dynamometers to determine the location of the force produced by the thrust of the arch ring.

Again, to determine the position of a body in space requires six independent quantities, three to locate the position of a point in it, and three to determine the amount it is rotated about each of three perpendicular axes. These quantities are also independent.

Therefore, in an experiment of this character, all six components of the abutment reaction must be observed and all six possible movements of the abutment guarded against.

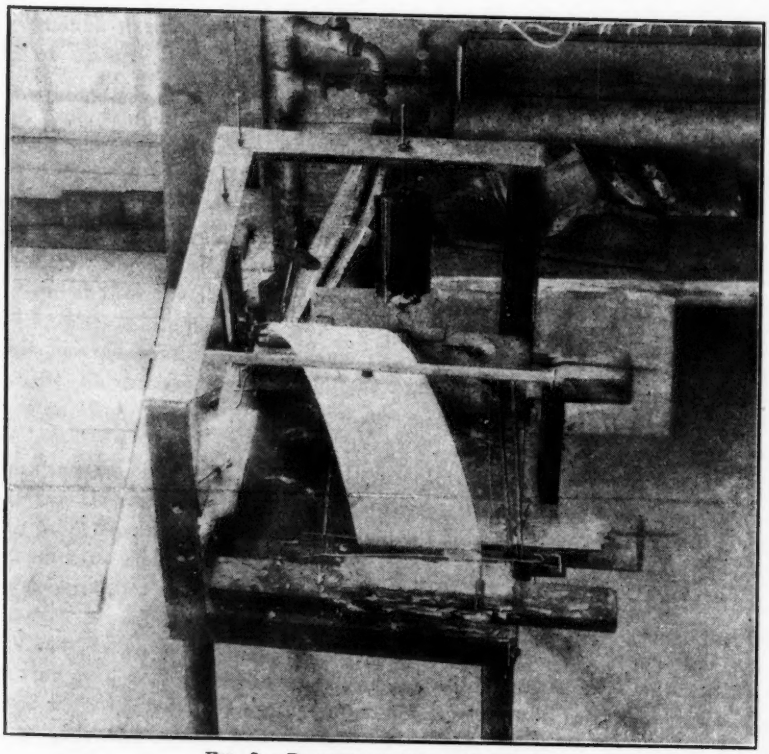

FIG. 2.—PLASTER OF PARIS MODEL.

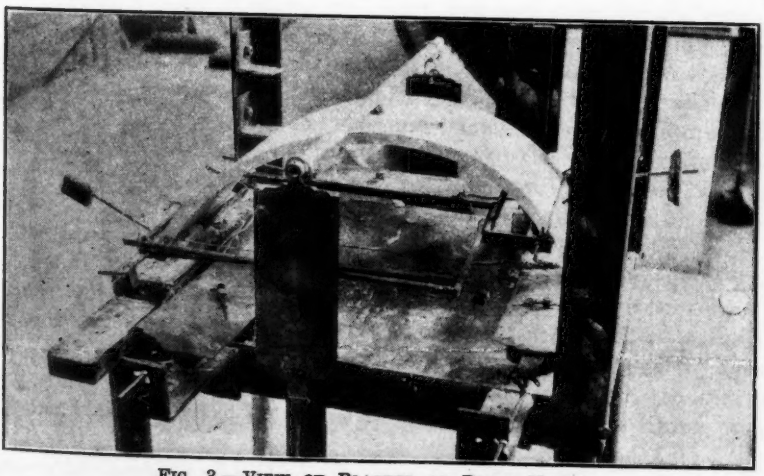

FIG. 3.—VIEW OF PLASTER OF PARIS MODEL.

Models

The models used were made of plaster of Paris mixed with 60% (by weight) of water. This mix was decided upon after a number of experiments with varying water contents. The value of E was found to vary greatly with the proportion of water used, being about 400 000 lb. per sq. in. when equal parts by weight of water and plaster (100%) were used and 1 100 000 lb. per sq. in. when one-half this quantity of water was taken. Thus, it is quite obvious that great care must be used to secure a uniform mixture. If the mix was too rich the rate of set was rapid and the tendency to lump was considerable. The result, however, was a much stronger material. It was finally decided that the least quantity of water that could safely be used, was 60%, by weight, of the plaster. For the sample of plaster of Paris used in this work this percentage gave ample time to mix the batch, work out all lumps, pour it into the mould, and remove the entrained air before the initial set. At the same time the resulting model was strong enough to withstand rough handling.

A mixture of plaster of Paris and water proved to be a very satisfactory material for the purpose of studying stresses in a model with the view of applying the information thus obtained to a study of stresses in a larger concrete structure. Being a cast material it has somewhat the same characteristics as concrete and when mixed with care is very uniform in quality. After being cast, a model can be sawed or cut with a chisel or plane. This property was very valuable in shaping the skew arch rings.

The ring was cast as the segment of a cylinder with an inside radius of 12 in. and an outside radius of 12¾ in. The form for the intrados consisted of a sheet of galvanized iron held in a semi-circular shape by a wooden frame. Two pieces of ¾ by ¾-in. rubber packing were laid on this form a distance apart that would insure sufficient material for trimming the completed model. Over these was placed a piece of galvanized sheet iron as a form for the extrados. This was fastened by bolts attached to the edges and screwed to braces over the wooden frames. Thus, the two pieces of sheet iron were held securely ¾ in. apart. The form was then turned completely over, and the arch was cast by pouring the soupy mixture in one side and letting it rise to the other side. A wire was threaded through the forms and worked back and forth to remove any air pockets that might be present. The sheet iron was greased to prevent the plaster from sticking to the forms. The arch was allowed to set 24 hours before the forms were removed.

Lines were drawn on the outer sheet of iron with a soft lead pencil. These lines were laid out as a rectangular grid before the sheet was bent over the forms, and the sheet was placed so that one set of these lines was parallel to the axis of the barrel of the arch. The crown and lines parallel to and 60° from it were particularly noted. When the forms were removed the lines were found plainly marked on the model and aided greatly in truing up and setting it.

The rubber packing when laid on the circular form did not lie in a plane, of course, so the model had to be trimmed. This was done by clamping a thin

board to the arch so that it was tangent at the crown line and skewed to the axis of the barrel. The model was then laid on a table with the plane of the board perpendicular to the table. Two lines were scribed on the plaster parallel to the table surface. These lines marked the two sides of the completed arch ring. By using an ordinary carpenter's saw with coarse teeth, the ring was sawed close to these lines and then trimmed down with a carpenter's jack-plane.

Thus, the completed model consisted of an arch ring of a cast material with a low modulus of elasticity and considerable strength. It was formed by the segment of a cylinder subtending an arc of 120° and was limited by two parallel planes, which make an angle of $(90 - \epsilon)$ with the axis of the barrel of the arch, ϵ being the angle of skew.

The lines 60° from the crown were used to set the collars for recording abutment deflections.

Apparatus

One of the fundamental assumptions in the arch theory is that the relative position of the abutments does not vary. In designing the testing apparatus it was necessary, therefore, to be assured of this condition and, at the same time, to be able to record the six components of the abutment reaction.

The arch ring proper was considered to lie between the two collars that were clamped to the arch ring. (See Figs, 2, 3, and 4.) Each of these collars had contact with the ring at four points by means of sharp screws resting against fiber washers. The washers rested on the arch ring at the centers of the four sides of a section of the ring. In order to be sure that the collars maintained the same position relative to each other, six independent points of contact had to be maintained. This was done by means of a system of levers. The left collar had attached thereto an arm that reached to within $\frac{5}{8}$ in. of another arm attached to the right collar. In this space was inserted a steel phonograph needle sharpened at both ends. To the needle was attached a stiff wire pointer, properly counterweighted so that the center of gravity lay at the center of the needle. The end of the pointer was in line with the axis of the needle. A short wire was attached to the nearest collar and the free end was adjusted so as to mark the initial position of the end of the pointer. By this device a movement, either vertical or horizontal, of one end of the needle relative to the other, was multiplied $\frac{17 \times 8}{5}$, or 27.2 times. To the right collar were attached two arms which also engaged phonograph needles in a similar manner. These two needles were placed so that their axes were in line. Thus, there were three needles, each recording two motions, making a total of six independent points of contact between the two collars. To show that they were independent it is only necessary to demonstrate that each needle could be moved either vertically or horizontally without affecting any of the other five settings.

A rotation of the right abutment relative to the left about a line joining the points farthest from their respective pointers of Needles Nos. 2 and 3 would give a vertical movement only to the pointer of Needle No. 1 without affecting the setting of the other two pointers. A result similar to that obtained for Needle No. 1 could be obtained for Needles Nos. 2 and 3. A horizontal rotation of the right abutment relative to the left about the vertical

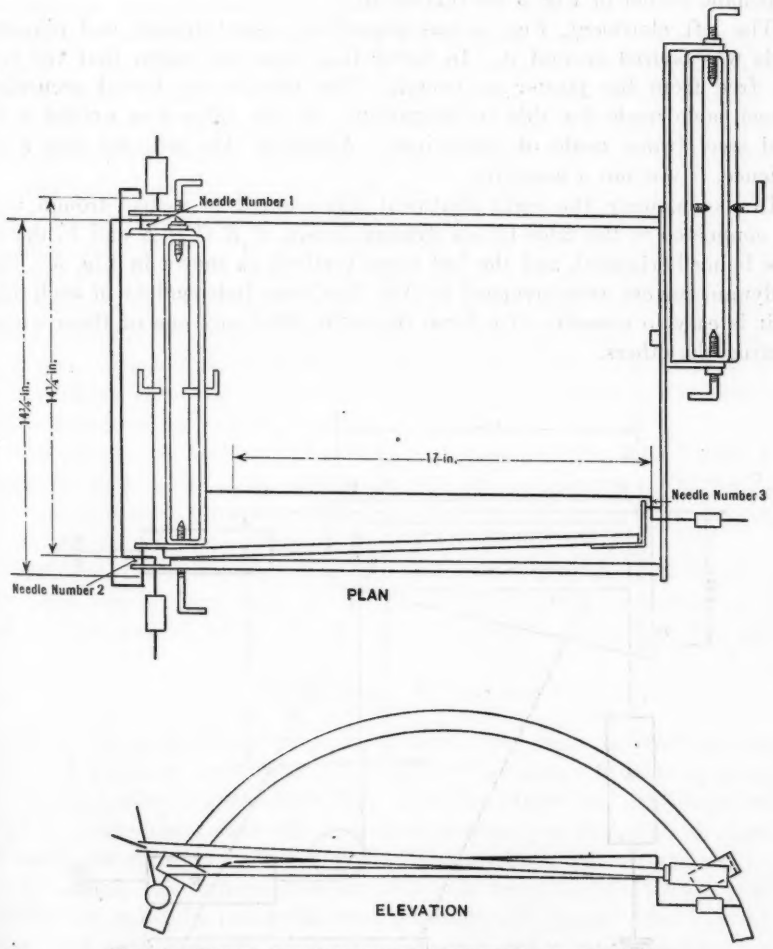

FIG. 4.—DEVICE FOR RECORDING MOVEMENTS OF ABUTMENT.

line formed by the intersection of the planes perpendicular to Needles Nos. 2 and 3 and passing through the one of their points in contact with the right abutment would give a horizontal movement to the pointer of Needle No. 1. Similarly, for Needles Nos. 2 and 3. For Needle No. 3, it is to be noted that the axis of rotation is at infinity. This does not affect the result.

The load used was a concentrated load placed at the center of the crown. The method of loading is shown in Figs. 2 and 3. A ½-in. cut washer was placed on the arch ring, and on this was placed a ½-in. nut, in which was a stud with the end rounded off. The pipe that was used as a beam to apply the load was flattened by filing slightly where it rested on the stud. The ends of the pipe carried milk scales for dynamometers. The load was applied by inducing a force on the scales with a hook-bolt and butterfly nut. (See lower right-hand corner of Fig. 3 for this detail.)

The left abutment, Fig. 3, was placed in a steel trough and plaster of Paris was poured around it. In doing this, care was taken that the collar was free from the plaster or trough. The trough was bolted securely to a steel table made for this investigation. To the table was welded a very rigid steel frame made of angle iron. Although this property was a convenience, it was not a necessity.

In like manner, the right abutment was secured to a steel trough which was connected to the table by six dynamometers, *A B C D E* and *F*, the first three being horizontal, and the last three vertical, as shown in Fig. 5. These six dynamometers were arranged so that they were independent of each other, for it is easy to conceive of a force that will affect any one of them without affecting the others.

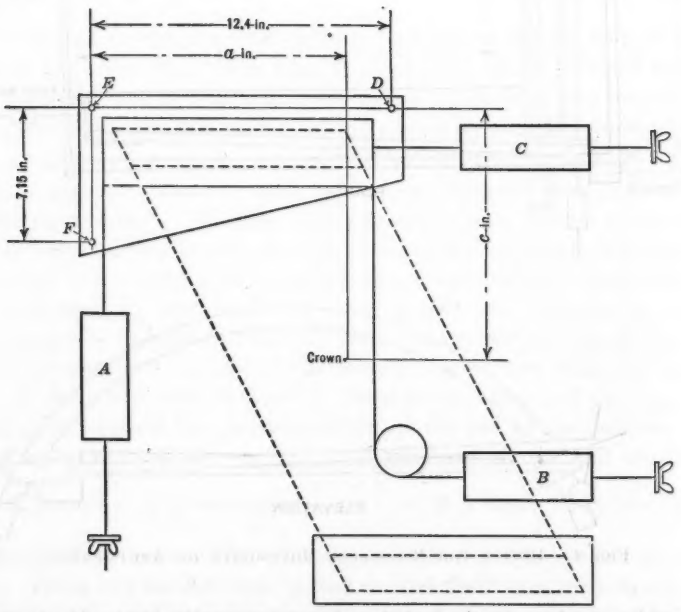

FIG. 5.—LOCATION OF DYNAMOMETERS WITH RESPECT TO CROWN OF ARCH.

The scales were adjusted after each increment of load was added until each of the three pointers was at its initial position; therefore, the collars remained in the same relative position to each other throughout the experiment and any

deflection due to lack of rigidity of the table, frame, troughs, or arch ring from trough to collar, was taken up while adjusting the scales.

Method of Performing Experiment

After the model was trued up, as previously described, the collars were placed in position on the marks drawn for them. The free trough was then blocked in place on the table, at the same elevation as the fixed trough and in a position that allowed the vertical scales to hang plumb. It was clamped securely and the model was placed in the troughs, leveled at the crown, and oriented so that the barrel of the arch was parallel to the plane through the axes of Scales *D* and *E*. The portions of the arch ring in the troughs were thoroughly wetted, after which plaster of Paris, mixed with 60% of water, was poured around them until the troughs were filled. When placed thus, the arch had no initial stress, except that due to its own weight. After the plaster had set 24 hours the needles were placed in position and adjusted. The loading beam was then placed across the top of the arch and the dynamometers were hung therefrom. Enough load was placed on the dynamometers to hold them steady. The load then on the crown was less than 10 lb. The dynamometers at the right abutment were tightened and the clamps removed. They were then adjusted until all the needle pointers were back to their initial position, both vertically and horizontally. This was taken as the zero crown load in constructing Figs. 8 to 12.

The crown load was added by small increments as shown in Tables 1 to 6, inclusive, and, after each increment, the dynamometers were re-adjusted. This was continued until the capacity of one of the dynamometers was reached, or the crown load exceeded its initial value by 50 lb.

Although the re-adjustment of the needles by applying loads to the dynamometers was in the nature of solving six equations in six unknowns by the cut and try method, after a little practice it could be done in a few minutes.

Preliminary Experiment

After the preliminary work to ascertain the most satisfactory mix for plaster of Paris, an experiment was considered necessary to gain an idea of the maximum vertical crown load that could be placed on the model without injury. From these data the testing apparatus was designed, or more particularly, the capacity required for the dynamometers was ascertained.

To obtain this information an arch was cast, the ends were placed in notches cut in a 2-in. board and were plastered into place. (See Fig. 6.) The board and arch were put on a platform scale and a pipe was placed on the crown parallel to the axis of the barrel of the arch. A load was applied to this pipe by means of a plank resting on a knife-edge and holding a vessel on one end and a counterweight on the other. The knife-edge was off the scale and the load was applied by pouring water in the vessel, thus tipping the plank on to the arch. It was measured by weighing the added load on the scales. The deflections were taken at the crown with an Ames dial. In this arch the

angle of skew was 22½° and the breadth measured parallel to the abutments was 10 in. Fig. 7 shows the arch after failure. Note that the breaks are parallel to the axis of the barrel. The results of this run are given in Fig. 8.

Comparison with Theory

The components of the crown reactions on a skew arch ring of the character of the models used have been determined for a vertical crown load using the elastic theory previously mentioned. This work is given in the Appendix. By taking the readings of the abutment dynamometers, as determined from Tables 1, 2, and 3, and from these, computing the stresses at the crown, a comparison can be made.

The crown reactions were computed from the following equations of equilibrium for the left half of the arch ring:

$$\left.\begin{aligned} T_x &= A + B \\ T_y &= D + E + F \\ T_z &= C \\ M_z &= c\,T_y - 7.15\,F - e\,T_x \\ M_x &= 12.4\,D - a\,T_y + d\,T_z \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, A, B, C, D, E, and F are the abutment dynamometer readings, and a = crown to plane, $E\,F$; b = width of arch ring; c = crown to plane, $D\,E$; d = elevation of crown above C; and e = elevation of crown above plane, $A\,B$, as defined in Tables 1, 2, and 3. The values of the dynamometer readings were taken from the slopes of the lines as plotted from these tables.

Fig. 9 was plotted from Table 1. A 50-lb. increase in the crown load would produce the following increases in the abutment dynamometer readings: A = 26.5 lb.; B = 15.5 lb.; C = 19.5 lb.; D = 10.3 lb.; E = 21.3 lb.; F = — 3.3 lb.; which with the following measurements: a = 11.125 in.; c = 15.00 in.; d = 8.31 in.; and e = 8.16 in., substituted in Equations (1) give T_x = 42.0 lb.; T_y = 27.3 lb.; T_z = 19.5 lb.; M_x = — 21.7 lb.; and M_z = 85.8 lb.

The theoretical values are: T_x = 40.4 lb.; T_y = 25.0 lb.; T_z = 15.7 lb.; M_x = — 18.1 lb.; and M_z = 61.2 lb.

Fig. 10 was plotted from Table 2. A distinct break occurs in the curves at the 30.5-lb. load, where Dynamometer F ran to zero and it became necessary to place a weight on the trough where it was attached to this dynamometer.

From the lower part of the curves for a 50-lb. increase: A = 12.4 lb.; B = 25.9 lb.; C = 21.0 lb.; D = 13.5 lb.; E = 18.2 lb.; and F = — 7.9 lb.

With a = 16.6 in ; c = 14.85 in.; d = 9.5 in.; and e = 9.34 in., Equations (1) gives, T_x = 38.3 lb.; T_y = 23.8 lb.; T_z = 21.0 lb.; M_x = — 28.2 lb.; and M_z = 51.2 lb.

From the upper part of the curves for a 50-lb. increase: A = 17.2 lb.; B = 22.3 lb.; C = 26.0 lb.; D = 11.4 lb.; E = 21.5 lb.; and F = — 7.5 lb.; which gives as values for the crown forces and moments: T_x = 39.5 lb.; T_y = 25.4 lb.; T_z = 26.0 lb.; M_x = — 33.3 lb.; and M_z = 65.3 lb.

The theoretical values are: T_x = 40.6 lb.; T_y = 25.0 lb.; T_z = 23.8 lb.; M_x = — 34.6 lb.; and M_z = 60.8 lb.

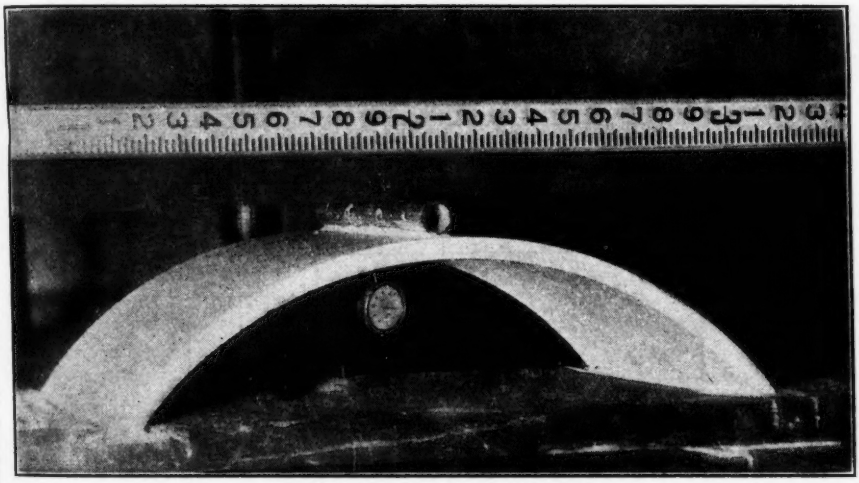

FIG. 6.—TEST ON UNLOADED ARCH.

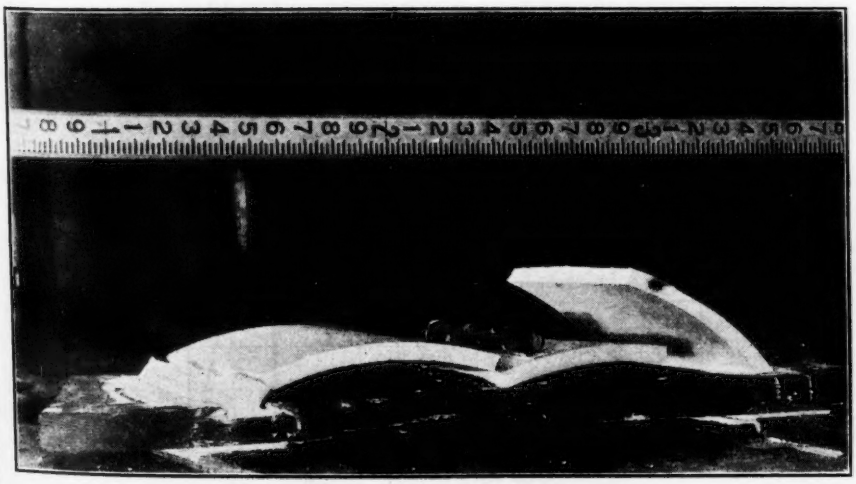

FIG. 7.—VIEW OF ARCH AFTER FAILURE.

Fig. 11 was plotted from Table 3. A 50-lb. increase in the crown load would produce the following: $A = 16.0$ lb.; $B = 27.2$ lb.; $C = 27.6$ lb.; $D = 15.0$ lb.; $E = 16.2$ lb.; and $F = -2.4$ lb. With $a = 17.0$ in.; $c = 15.1$ in.; $d = 9.26$ in.; and $e = 9.1$ in., Equations (1) gives: $T_x = 43.2$ lb.; $T_y = 28.8$ lb.; $T_z = 27.6$ lb.; $M_x = -48.0$ lb.; and $M_z = 58.9$ lb.

The theoretical values are: $T_x = 40.8$ lb.; $T_y = 25.0$ lb.; $T_z = 31.0$ lb.; $M_x = -43.3$ lb.; and $M_z = 59.8$ lb.

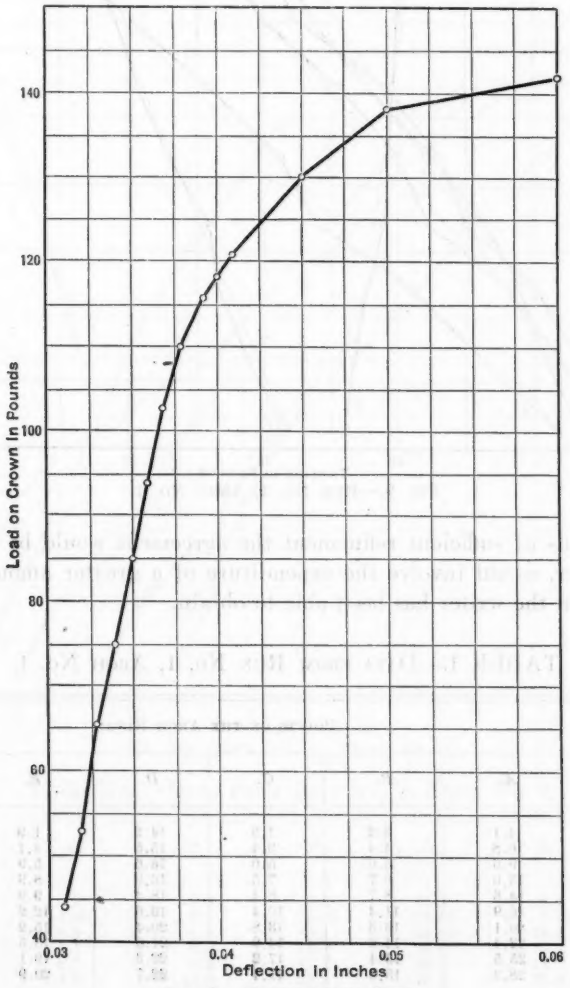

FIG. 8.—PRELIMINARY RUN.

Although the agreement of the theoretical and the observed values is not as close as could be desired, considering the many sources of error in the experiments, it is close enough to convince one that the theory is a reliable method of obtaining stresses in a skew arch ring and the writer feels that

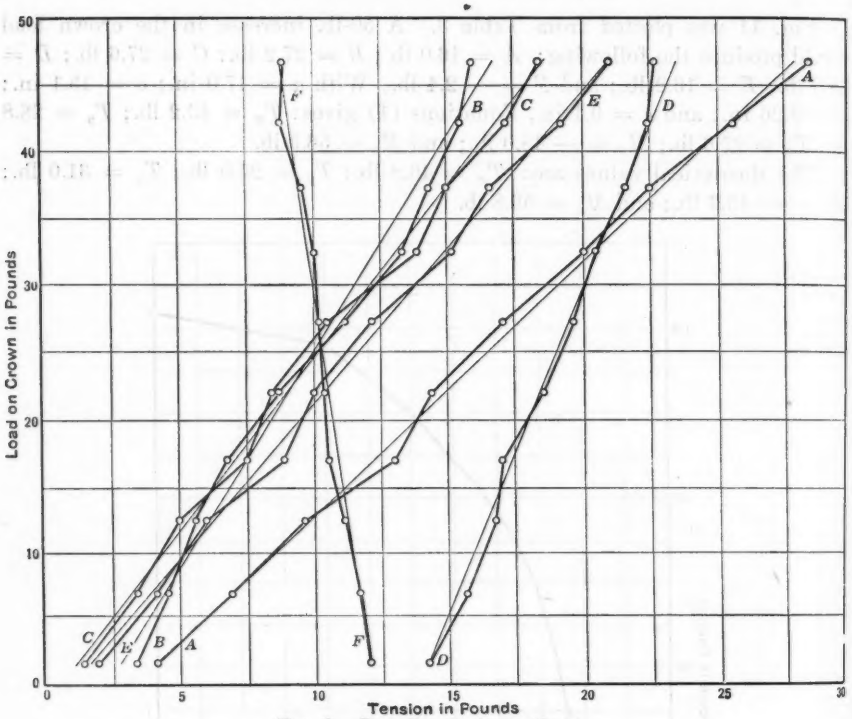

FIG. 9.—RUN NO. 1, ARCH NO. 1.

with apparatus of sufficient refinement the agreement would be closer. This work, however, would involve the expenditure of a greater amount of money and time than the writer has been able to obtain.

TABLE 1.—DATA FROM RUN NO. 1, ARCH NO. 1.

Crown load.	POINTS ON THE ARCH RING:					
	A.	*B.*	*C.*	*D.*	*E.*	*F.*
1.7	4.1	3.2	1.3	14.2	1.9	12.0
7.0	6.8	4.4	3.4	15.5	4.1	11.6
12.0	9.6	5.6	5.0	16.6	5.9	11.1
17.1	13.0	6.7	7.5	16.9	8.9	10.5
22.0	14.3	8.7	8.4	18.4	9.9	10.3
27.4	16.9	11.4	10.4	19.6	12.2	10.2
32.6	20.1	13.3	13.8	20.4	15.2	10.0
37.4	22.4	14.2	14.9	21.6	16.5	9.6
42.3	25.5	15.4	17.2	22.3	19.1	8.8
46.9	28.3	15.8	19.4	22.7	20.9	9.0

NOTE.—a = 11.125 in.; b = 9.75 in.; c = 15.00 in.; d = 8.31 in.; e = 8.16 in.; and angle of skew = 22 degrees.

EFFECT OF THE ABUTMENT MOVEMENTS

The abutment of the arch with a 31° skew was moved in a horizontal direction without rotation parallel to the axis of the barrel of the arch, in order to

test the effect on the abutment reactions of such a movement. Unfortunately, no readings were taken to ascertain the amount of this movement. Needles Nos. 1 and 2 were kept in position while the pointer of Needle No. 3 was moved horizontally by varying the load on the dynamometers. Rotation of the abutments was also prevented. From Table 4 and Fig. 12 the tendency can be easily noted. As Dynamometer *C* was released in order to keep the span of the arch a constant, Dynamometer *B* received load from Dynamometer *A*, while Dynamometer *D* received load from Dynamometers *E* and *F*.

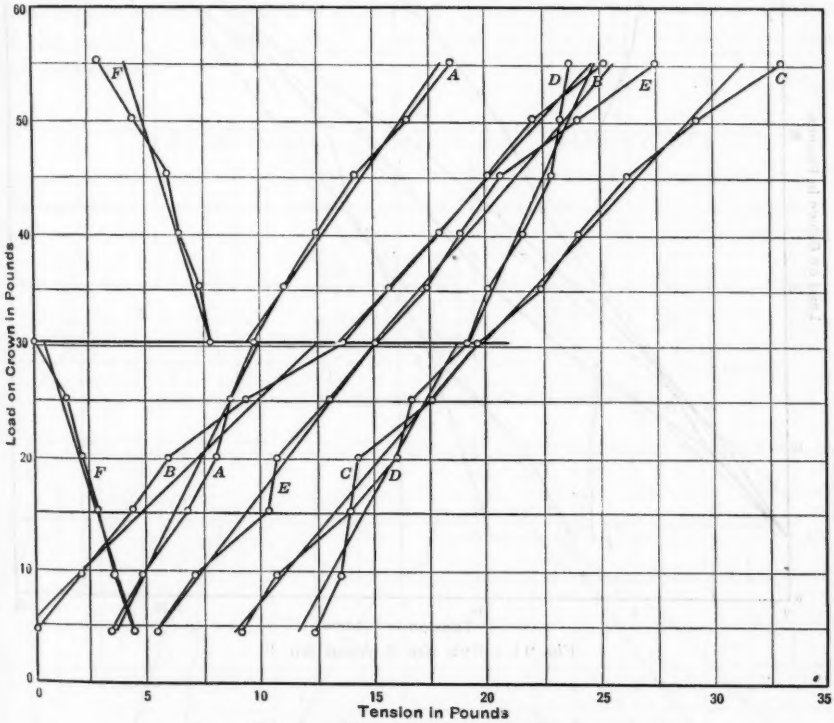

FIG. 10.—RUN NO. 2, ARCH NO. 2.

TABLE 2.—DATA FROM RUN NO. 2, ARCH NO. 2.

Crown load.	*A*	*B.*	*C.*	*D.*	*E.*	*F.*
4.6	3.5	—0.2	9.1	12.3	5.5	4.5
9.85	4.85	2.1	10.75	13.55	7.15	3.6
15.5	6.9	4.4	14.1	14.05	10.4	2.8
20.2	8.1	5.9	14.3	16.0	10.7	2.1
25.3	8.7	9.5	17.5	16.7	13.1	1.5
30.5	9.8	14.0	19.6	19.2	15.2	0.0
30.5	9.8	14.0	19.6	19.2	15.2	7.9
35.3	11.1	15.7	22.3	20.0	17.3	7.3
40.2	12.5	17.9	24.0	21.6	18.8	6.4
45.4	14.2	20.0	26.1	22.8	20.5	6.0
50.4	16.4	22.0	29.3	23.2	23.9	4.5
55.3	18.4	25.0	32.9	23.6	27.3	3.0

NOTE.—a = 16.60 in.; b = 8 in.; c = 14.85 in.; d = 9.50 in.; e = 9.34 in.; and angle of skew = 31 degrees.

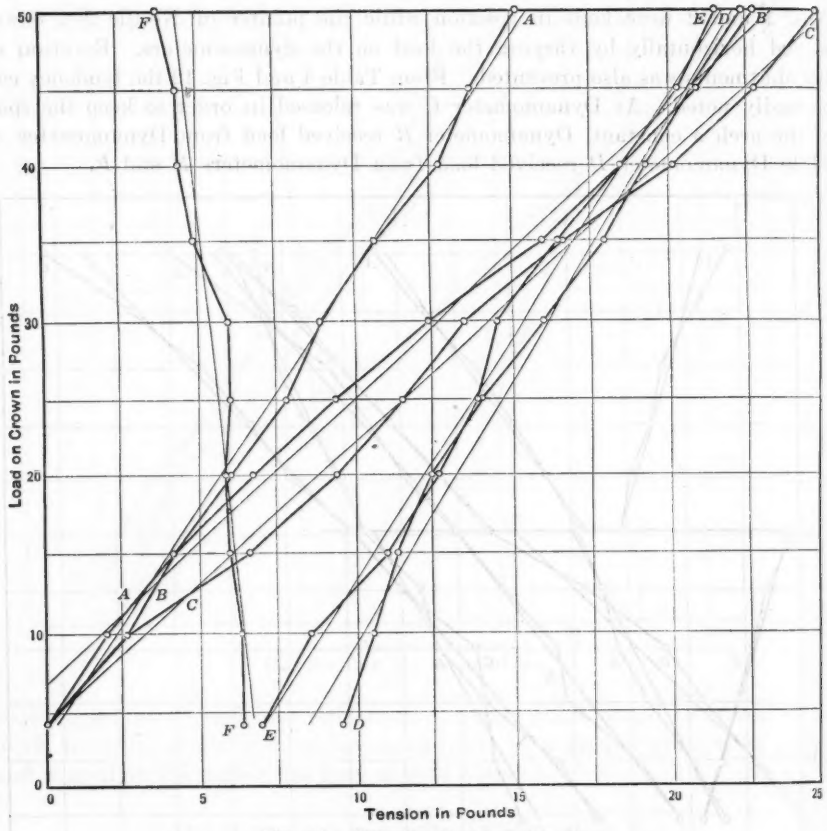

FIG. 11.—RUN NO. 3, ARCH NO. 3.

TABLE 3.—DATA FROM RUN NO. 3, ARCH NO. 3.

Crown load.	*A.*	*B.*	*C.*	*D.*	*E.*	*F.*
4	0.0	0.0	0.0	9.5	6.9	6.3
10	2.0	2.7	2.7	10.5	8.6	6.3
15	4.2	4.2	6.5	11.3	11.0	5.9
20	6.0	6.7	9.4	12.5	12.6	5.9
25	7.8	9.4	11.5	14.0	13.7	6.0
30	9.0	12.3	13.5	16.0	14.6	5.9
35	10.7	16.0	16.7	18.0	16.6	4.9
40	12.7	18.6	20.2	19.3	18.6	4.4
45	13.8	21.0	22.8	20.7	20.3	4.4
50	15.3	22.8	24.9	22.5	21.4	3.8

NOTE.—a = 17.00 in.; b = 9 in.; c = 15.10 in.; d = 9.26 in.; e = 9.10 in.; and angle of skew = 37 degrees.

A third arch was put in the testing frame and a load of about 30 lb. was placed on the arch crown. This arch had an angle of skew of 37° and a length of abutment of 9 in. The abutment was moved parallel to itself without rotation and in a direction to widen the span, horizontally but at right angles to the barrel of the arch. Table 4 and Fig. 12 give the effect of this movement. This is a possible direction in which the arch may slip under load. It is interesting to note that the horizontal force, *B*, becomes zero, or that the abutment force is very near the point of the arch. Table 5 is produced by a movement similar to the one producing Table 3; that is, parallel to the axis of the barrel of the arch. It is interesting to compare the results of these two movements.

TABLE 4.—DATA FROM RUN NO. 4, ARCH NO. 2.*

Crown load.	*C.*	*A.*	*B.*	*D.*	*E.*	*F.*
50	16.5	7.2	23.2	21.8	17.0	5.0
50	14.0	4.5	25.2	24.0	15.8	4.0
50	12.0	2.5	28.2	26.0	13.8	3.7
50	9.0	1.0	31.3	28.1	11.5	3.2
50	6.0	0.0	34.3	30.2	10.0	2.5
550	3.0	−1.2	37.2	32.6	8.3	2.2
50	0.0	−2.2	39.1	35.1	6.1	1.7

* Abutment moved horizontally and parallel to barrel of arch.

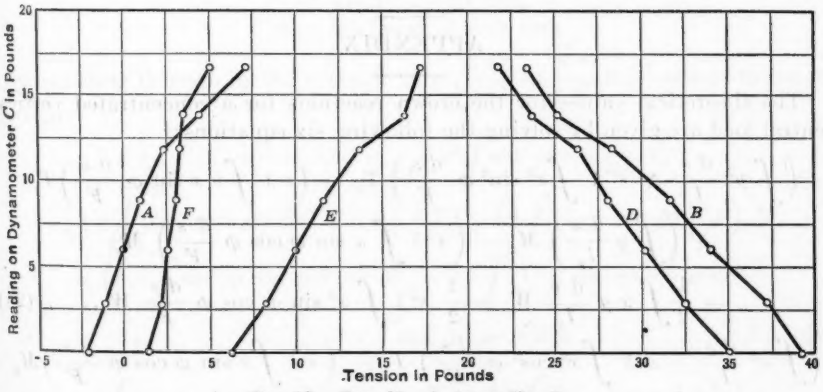

FIG. 12.—RUN NO. 4, ARCH NO. 2.

TABLE 5.—DATA FROM RUN NO. 5, ARCH NO. 3.*

Crown load.	*A.*	*B.*	*C.*	*D.*	*E.*	*F.*
30.2	9.1	6.3	8.8	16.6	13.2	4.9
30.0	11.4	0.0	6.6	15.1	12.7	5.0

* Abutment moved horizontally and perpendicular to barrel of arch.

Conclusions

Although the apparatus for recording abutment movements could be improved a great deal by being made lighter and a method could be devised for controlling the pressure on the needles, it is the writer's opinion that the following conclusions are warranted:

(1) If the abutment moves in a known direction and amount, the stresses are redistributed in a manner that can be foreseen from the elastic theory.* They may cause the heavier load on one or the other end of the abutment, depending on the direction of slipping.

(2) The force represented by T_z, or Dynamometer C, must be provided for in the design of a skew arch ring.

(3) The obtuse corner of the abutment does not necessarily carry any heavier load than the acute corner, and may carry less.

(4) The discrepancies between the values obtained by the arch theory and those obtained by experimentation are within the range of experimental error so that this set of experiments supports the theory.

Acknowledgment

Appreciation is extended to C. E. Magnusson, M. Am. Soc. C. E., for obtaining funds from the University of Washington Experiment Station with which to make the necessary apparatus and models to conduct these tests. Also to W. R. Engstrom, Jun. Am. Soc. C. E., for obtaining a large percentage of the data herein given.

APPENDIX

The theoretical values for the crown reactions for a concentrated vertical central load are given by solving the following six equations:†

$$\left(\int y^2 \frac{d\,s}{I_z} + \varepsilon^2 \lambda \int x^2 \sin^2 \phi \frac{d\,s}{F}\right) T_x - \left(\varepsilon \lambda \int u\,x \sin \phi \frac{d\,s}{F}\right) T_z + \left(\int y \frac{d\,s}{I_z}\right) M_z - \left(\varepsilon \lambda \int x \sin \phi \cos \phi \frac{d\,s}{F}\right) M_x = \frac{1}{2}\int x\,y \frac{d\,s}{I_z} W + \frac{1}{2} \varepsilon^2 \lambda \int x^2 \sin \phi \cos \phi \frac{d\,s}{F} W \quad \ldots\ldots (2a)$$

$$\left(\int x^2 \frac{d\,s}{I_z} + \varepsilon^2 \lambda \int x^2 \cos^2 \phi \frac{d\,s}{F}\right) T_y - \left(\varepsilon \lambda \int x \sin \phi \cos \phi \frac{d\,s}{F}\right) M_y = -\frac{1}{2}\int x^2 \frac{d\,s}{I_z} W - \frac{1}{2} \varepsilon^2 \lambda \int x^2 \cos^2 \phi \frac{d\,s}{F} W \quad \ldots\ldots (2b)$$

$$\left(-\varepsilon \lambda \int u\,x \sin \phi \frac{d\,s}{F}\right) T_x + \left(\lambda \int u^2 \frac{d\,s}{F}\right) T_z + \left(\lambda \int u \cos \phi \frac{d\,s}{F}\right) M_x = -\frac{1}{2} \varepsilon \lambda \int u\,x \cos \phi \frac{d\,s}{F} W \quad \ldots\ldots\ldots\ldots\ldots (2c)$$

$$\left(\int y \frac{d\,s}{I_z}\right) T_x + \left(\int \frac{d\,s}{I_z}\right) M_z = \frac{1}{2} \int x \frac{d\,s}{I_z} W \quad \ldots\ldots (2d)$$

* Compare the results of Tables 4 and 5 with the statements in *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 675.

† *Transactions*, Am. Soc. C. E., Vol. LXXXVII, pp. 617–618 and pp. 643–644.

$$\left(-\varepsilon\lambda\int x\sin\phi\cos\phi\frac{ds}{F}\right)T_y+\left(\lambda\int\sin^2\phi\frac{ds}{F}\right)M_y$$
$$=\frac{1}{2}\varepsilon\lambda\int x\sin\phi\cos\phi\frac{ds}{F}W\ldots\ldots\ldots\ldots\ldots\ldots(2e)$$

$$\left(-\varepsilon\lambda\int x\sin\phi\cos\phi\frac{ds}{F}\right)T_x+\left(\lambda\int u\cos\phi\frac{ds}{F}\right)T_z$$
$$+\left(\lambda\int\cos^2\phi\frac{ds}{F}\right)M_x=-\frac{1}{2}\varepsilon\lambda\int x\cos^2\phi\frac{ds}{F}W\ldots\ldots(2f)$$

The solution of Equations (2*b*) and (2*e*) is obviously $T_y=-\frac{1}{2}$ and $M_y=0$, and no further consideration of these equations is necessary.

As the arch consists of the segment of a circular cylinder, from the properties of a circle the following substitutions may be made:

$$x=r\sin\phi$$
$$y=r(1-\cos\phi)$$
$$u=y=r(1-\cos\phi)$$
$$v=x=r\sin\phi$$
$$ds=r\,d\phi$$
$$\frac{1}{I_x}=\frac{12}{b\,d^3}=4\times\frac{3}{b\,d^3}$$
$$\frac{\lambda}{F}=\frac{9}{b\,d^3}=3\times\frac{3}{b\,d^3}$$
b = width of arch ring.
d = thickness of arch ring.

After dividing through by the common term, $\frac{3\,r^2}{b\,d^3}$, or $\frac{3\,r}{b\,d^3}$, this reduces the equations to:

$$\left(4\int(1-\cos\phi)^2\,d\phi+3\,\varepsilon^2\int\sin^4\phi\,d\phi\right)r\,T_x$$
$$-\left(3\,\varepsilon\int(1-\cos\phi)\sin^2\phi\,d\phi\right)r\,T_z+\left(4\int(1-\cos\phi)\,d\phi\right)M_z$$
$$-\left(3\,\varepsilon\int\sin^2\phi\cos\phi\,d\phi\right)M_x=2\,r\int(1-\cos\phi)\sin\phi\,d\phi$$
$$+\frac{3}{2}\,\varepsilon^2\,r\int\sin^3\phi\cos\phi\,d\phi\ldots\ldots\ldots\ldots\ldots\ldots(3a)$$

$$\left(-3\,\varepsilon\int(1-\cos\phi)\sin^2\phi\,d\phi\right)r\,T_x+\left(3\int(1-\cos\phi)^2\,d\phi\right)r\,T_z$$
$$+\left(3\int(1-\cos\phi)\cos\phi\,d\phi\right)M_x$$
$$=-\frac{3}{2}\,\varepsilon\,r\int(1-\cos\phi)\sin\phi\cos\phi\,d\phi\ldots\ldots\ldots\ldots(3c)$$

$$\left(4\int(1-\cos\phi)\,d\phi\right)r\,T_x+\left(4\int d\phi\right)M_z=2\,r\int\sin\phi\,d\phi\ldots(3d)$$

$$\left(-3\,\varepsilon\int\sin^2\phi\cos\phi\,d\phi\right)r\,T_x+\left(3\int(1-\cos\phi)\cos\phi\,d\phi\right)r\,T_z$$
$$+\left(3\int\cos^2\phi\,d\phi\right)M_x=-\frac{3}{2}\,\varepsilon\,r\int\sin\phi\cos^2\phi\,d\phi\ldots\ldots(3f)$$

Integrating and substituting the limits of $\frac{\pi}{3}$ and 0, and reducing terms:

$$\left(\frac{4\pi}{3} - 4\sqrt{3} + \frac{\sqrt{3}}{2} + \frac{2\pi}{3} - \frac{9\sqrt{3}}{64}\varepsilon^2 - \frac{9\sqrt{3}}{32}\varepsilon^2 + \frac{3\pi}{8}\varepsilon^2\right) r\,T_x + \left(\frac{3\sqrt{3}}{8}\varepsilon - \frac{\pi}{2}\varepsilon + \frac{3\sqrt{3}}{8}\varepsilon\right) r\,T_z + \left(\frac{4\pi}{3} - \frac{4\sqrt{3}}{2}\right) M_z - \left(\frac{3\sqrt{3}}{8}\right)\varepsilon\,M_x = -\left(2r + \frac{3}{2}r - \frac{27}{64}\varepsilon^2 r - 4r\right)\frac{1}{2} \quad \ldots\ldots(4a)$$

$$\left(\frac{3\sqrt{3}}{8} - \frac{\pi}{2}\varepsilon + \frac{3\sqrt{3}}{8}\varepsilon\right) r\,T_x + \left(\pi - 3\sqrt{3} + \frac{3\sqrt{3}}{8} + \frac{\pi}{2}\right) r\,T_z + \left(\frac{3\sqrt{3}}{2} - \frac{3\sqrt{3}}{8} - \frac{\pi}{2}\right) M_x = -\left(\frac{9}{8}\varepsilon r + \frac{1}{8}\varepsilon r - \varepsilon r\right)\frac{1}{2} \quad \ldots(4c)$$

$$\left(\frac{4\pi}{3} - 2\sqrt{3}\right) r\,T_x + \frac{4\pi}{3} M_z = (-2r + 4r)\frac{1}{2} \quad \ldots\ldots(4d)$$

$$\left(-\frac{3\sqrt{3}}{8}\right)\varepsilon\,r\,T_x + \left(\frac{3\sqrt{3}}{2} - \frac{3\sqrt{3}}{8} - \frac{\pi}{2}\right) r\,T_z + \left(\frac{3\sqrt{3}}{8} + \frac{\pi}{2}\right) M_x = \left(\frac{1}{8}\varepsilon r - \varepsilon r\right)\frac{1}{2} \quad \ldots\ldots(4f)$$

Substituting the numerical values, these equations reduce to:

$$(0.2210 + 0.4474\,\varepsilon^2)\,r\,T_x - 0.2718\,\varepsilon\,r\,T_z + 0.7247\,M_z - 0.6495\,\varepsilon\,M_x = (0.2500 + 0.2110\,\varepsilon^2)\,r \quad \ldots\ldots(5a)$$

$$-0.2718\,\varepsilon\,r\,T_x + 0.1658\,r\,T_z + 0.3777\,M_x = -0.1250\,\varepsilon\,r \quad \ldots\ldots(5c)$$

$$0.7247\,r\,T_x + 4.1888\,M_z = 1.0000\,r \quad \ldots\ldots(5d)$$

$$-0.6495\,\varepsilon\,r\,T_x + 0.3777\,r\,T_z + 2.2203\,M_x = -0.4375\,\varepsilon\,r \quad \ldots\ldots(5f)$$

the solutions of which are:

$$2\,T_x = \frac{2.0217 + 0.0689\varepsilon^2}{1.2550 + 0.0148\varepsilon^2}$$

$$2\,T_z = \frac{1.9620 + 0.0948\varepsilon^2}{1.2550 + 0.0148\varepsilon^2}\,\varepsilon$$

$$2\,M_x = \frac{-0.2340 - 0.0017\varepsilon^2}{1.2550 + 0.0148\varepsilon^2}\,\varepsilon\,r$$

$$2\,M_z = \frac{0.2495 - 0.0048\varepsilon^2}{1.2550 + 0.0148\varepsilon^2}\,r$$

TABLE 6.—ARCH DATA FOR $r = 12\frac{3}{4}$ INCHES.

Skew angle.	22°	31°	37°
ε	0.404	0.600	0.754
T_x	0.808	0.812	0.816
T_z	0.314	0.475	0.619
M_x	−0.361	−0.691	−0.866
M_z	1.224	1.216	1.197

With $r = 12\frac{3}{4}$ in., these values, for a unit central load, are as given in Table 6.

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PAPERS AND DISCUSSIONS

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INLETS ON SANDY COASTS

By Earl I. Brown,* M. Am. Soc. C. E.

Synopsis

Wave action on sandy coasts results in the suspension and transportation of material in what is known as littoral drift. This drift is powerfully affected by the strong hydraulic currents running through the inlets as the ocean tide rises above, or falls below, the mean level of the basin. The net results of the action of the currents on the drift are measured by the relative magnitudes of the currents and of the drift factors.

The ultimate effect of the combined action of the littoral drift and inlet currents, if uncontrolled, and if the ratio of drift to volume of water be too large, is to fill the interior basin with deposited material and to close the inlet. Some of the intermediate effects are to cause rapid and sometimes extensive physical changes in the beach adjacent to the inlet to the detriment of local improvements by man. The beaches and beds of bays and tidal channels, being mobile, readily respond to changes in current velocity, which control erosion or sedimentation in them.

Therefore, there is an intimate relation between the size of the interior bay and that of the inlet. A reduction in the size of the bay must lead to a reduction in the size of the inlet. The same result may be produced by an increase in the frictional resistance to the flow of currents between the bay and the inlet, causing the bay to be incompletely filled by the tidal currents. Due to the play between effects of drift and of currents, inlets may migrate or may divide as well as close.

Means can be found to stabilize inlets disposed to migrate, or to divide, by permanent works on the beach or by changes in the interior channels and bays. In many cases the volume of tidal flow through the inlet can be brought to the full capacity of the interior bay by such means, or by dredging. Permanent stone jetties are considered to be the most reliable means of controlling inlets.

Note.—Written discussion on this paper will be closed in August, 1928.

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Introduction

Exclusive of large bays and gulfs, peninsulas and capes, some of the most important features of a coast line are those breaks in its continuity which may be broadly classified as estuaries and inlets.

An estuary may also be the mouth of a large river, but it is characterized usually by being funnel-shaped, its opening to the sea being very wide, admitting the tidal wave for its full width, and causing the wave to increase in height or heap up as it travels inward, due to rising bottom or to converging banks, or to both. The inlet on a sandy coast generally has banks roughly parallel, connects two larger bodies of water, is usually small with respect to the interior basin, and admits but little of the tidal wave into that basin.

The characteristics of such openings as estuaries, permitting propagation of the tidal wave practically unchanged in form, have been treated of by many writers and will not be considered herein. The writer proposes to treat only of inlets of that class which are too small and too short to permit propagation of the full tidal wave, but through which the flow may be classed as hydraulic, that is, due to the difference of level between the water inside the inlet and that outside.

The South Atlantic Coast consists mainly of a strip of sandy earth immediately fronting on the ocean; bounded on the back by bays or sounds, or frequently by more or less extensive areas of salt marsh, which, in turn, front on the bays or sounds and through which numerous inlets afford channels for the flow of water between the bays and the sea. This, in general, describes the entire stretch of ocean front from Sandy Hook to Florida.

Sandy coasts present many difficulties to the student of problems pertaining to coast erosion and coast protection. Those difficulties are greatly magnified where there are inlets and where changes in the coast line are most rapid and most marked.

A study of the inlets from the standpoint of beach stability or change is a matter of much concern to pleasure resorts located on this coast, and must extend beyond the limits that would serve if navigation were the major subject under discussion. An inlet may be of no consequence to the navigator because of its tortuous, shoal, or shifting channel, and yet may exert a profound and far-reaching effect on the beaches immediately adjacent to it. The beach on and near the inlets is subject, not only to the same forces that raise or wash down the beach on the ocean front remote from the inlets, but in addition to the scour or deposit of material caused by the swift currents which are generated by the difference in elevation between the water of the bay and the water of the ocean.

The inlets are the critical points on the coast line with respect to the rate of physical change. It is there that such changes are most marked, many sections at points remote from the inlets having undergone very little or no change in the past eighty years or more, as shown by comparison of surveys made at various times. The inlets shift in position or greatly change their dimensions, and many have closed entirely within comparatively recent years, while new ones have opened.

This variability affects all industries dependent on the interior lagoons as fishing, oystering, clamming, and navigation, while vested property and other rights on the beaches are endangered and frequently destroyed as a result of such changes. To discover some of the laws which control inlet flow and govern the other natural forces which act so powerfully on the beaches at inlets, and to collect and formulate general principles and rules underlying the action of inlets for the assistance and guidance of those attempting to control or to improve inlets, are the main objects of this study.

Wave Action on Sea Beaches, South Atlantic Coast

Northeast and in lesser degree southeast storms cause a washing down of the beach. These storms raise the water to abnormal heights and, in addition, are accompanied by high waves. Therefore, they reach sections of the beach ordinarily far above the limit of the waves. Sand is dislodged, and the great changes in the water level generate longshore currents of considerable velocity, which transport large quantities of sand. This condition is aided, of course, by the oblique angle at which the waves generated by these northeast and southeast storms strike the beach.

In general, the material comprising the beaches is sand of a greater or less degree of fineness, and comminuted fragments of shells. It is driven back and forth and up and down the beach, with generally a resultant motion in one direction and the ultimate destination of being placed in solution or in suspension, and carried out to sea and deposited on the ocean bed.

Along sandy beaches, therefore, material is transported by the waves and the currents, and sometimes also by the winds. The waves act upon the bottom and set in motion those materials covering it over a greater or less width, according to local circumstances. The materials thus taken up are pushed toward the beach by the predominance which the direct current of the waves in general has over the return current. When the waves are normal to the beach—a matter of very rare occurrence—the material oscillates along the line of greatest slope, but without advancement laterally.

Most often the waves encounter the beach at a certain angle; they then project the material obliquely and it runs down toward the sea with the return wave conforming to the line of greatest slope. It is then re-seized and thrown up anew by the following wave, in such manner that it advances by successive movements in the same direction as the propagation of the waves.

The progress of the material also undergoes oscillations, for the wind does not always blow from the same direction. Following the direction of the waves, the particles go in one direction or another. However, since the waves come most often from some one direction there is finally a definite movement of transport in that direction. When the materials are subjected not only to the action of the waves, but also to that of the currents and the winds, their final translation is the resultant of the partial actions due to the diverse causes.

These causes and results of transport of material are magnified during storms, not only because the action of the waves on the bottom is then more powerful, but also because the heavy storms at sea, by urging the waters shore-

ward, provoke littoral currents the action of which, adding itself to that of the ordinary currents, is often sufficient to sweep the bottom clean. In any event it provides an increase in power to transport the material in suspension.

When, in traveling along the beach, the particles reach a large bay or other opening, or even an abrupt change in direction of the coast line, they may form into a point jutting out from the bank by which they approach, forming a sand spit. The ebb current then is inflected toward the opposite bank which may find itself attacked, if the opening be narrow.

If the bay be small and shallow, the spit tends to form a barrier and to close it up completely, but the ebb tide, aided by the fresh-water discharge, if there be any, maintains in the barrier a breach by which the alternating movement of the tide is made. The ebb, by scouring out the particles which tend to obstruct the breach, in what is termed the channel, forms before it a rim or cone of deposit, which receives the name of "bar". The current does not surmount the bar uniformly; it cuts out one or more grooves, which constitute passes, in which is found the greatest depth of water. The banks on either side of the passes are called "flats" or "shoals".

The passes vary in position after a storm or a freshet, which may have changed the relief of the bar. Under the action of the travel of the moving particles, called "littoral drift", the passes tend, moreover, to become displaced by moving in the same direction.

Littoral Drift

The travel of the littoral drift depends then on the lay of the coast with respect to the dominant direction of the waves, and, consequently, of the winds. According as the particles are carried more or less rapidly, the beach is denuded or built up. A mean regimen becomes established on every shore.

It may happen then that the coast has a bay or a cape the bisecting line of which may agree more or less closely with the prevailing winds. In such a case the drift may move in opposite directions on opposite sides, converging or diverging, as the case may be.

This littoral drift, in passing an inlet at flood tide, may be caught up by it and carried into the interior waters, where, as the velocity of the incoming tide decreases and stops, it may be deposited, the larger and coarser particles first and the smaller and lighter ones farthest within. The operation is then reversed by the ebb tide, which, as it progressively gathers strength, picks up the deposited material or part of it and returns it to the sea. Some of the deposited material is always left behind, and as it accumulates it forms bars through which channels or passes of varying size and depth also form to connect the interior and exterior waters. This is called the "inner bar". A similar bar is also formed just outside the inlet, where the outflowing ebb tide loses its velocity sufficiently to drop its burden of material.

This "outer bar", being subject to the action of the currents and waves of the sea, generally does not increase beyond a definite stage; the inner bar, however, being subject to much milder forces in the bay, continues to increase indefinitely, even to the complete filling of the interior waters by the formation of marshes and the closure of the inlet. While this progressive dete-

rioration of interior bays and sounds on sandy shores may not be a rapid process, as measured by the span of human life, yet geologically it is a very rapid process, many of them showing a marked deterioration within the memory of living persons.

Littoral drift is, therefore, a matter of prime importance in the study of an inlet, and is worthy of further consideration. Drifting may occur along the beach slope itself, termed "beach drifting", due to the swash of the waves advancing obliquely up the slope and returning back directly down the steepest slope, resulting in a loose particle on the beach pursuing a zigzag or parabolic course along the shore, as already described. Since smaller particles are more easily moved by the swash and are carried farther along at each step than larger particles, their progress along the beach is more rapid.

It is evident that the direction of beach drifting is independent of the direction of any longshore current, and is dependent only on the direction from which the wave impinges on the beach. This direction is, of course, not constant, and hence beach drifting will sometimes be in one direction along the beach and sometimes in the other. The direction of the resultant movement will depend on many factors, such as prevailing winds, relative intensity of winds, fetch of the ocean, depth of the foreshore, and the direction from which ground swells approach the shore.

As distinguished from beach drifting, "longshore drifting" is the transportation of particles which takes place in the shallow water seaward of the beach. To effect such transportation, a movement of the water or a current must exist. Therefore, the various ocean currents that may be effective in causing longshore drifting will be considered.

Ocean Currents

On approaching the shore line, the oscillating motion of waves becomes wholly or partly converted into a translating motion. Since waves of translation will cause a piling up of the water against the coast above the normal sea level, an accumulation of such excess water will be relieved only by the setting up of hydraulic currents, in the form of an undertow if the escape is along the bottom, or of a longshore current if the escape is laterally.

Tidal Currents.—Another type of ocean current is the tidal current. The passage of the tidal wave gives rise to corresponding movements of a particle of water, in both horizontal and vertical planes, each movement resembling a very elongated ellipse. As the tidal wave approaches, a particle of water over a sloping sea bottom near the shore will rise in the vertical plane and at the same time move shoreward, and, after the passage of high water, will begin to fall and soon to retreat from the shore, while in plan it will revolve in an elliptical orbit, of greater or less eccentricity.

This motion depends on the configuration of the shore line. At or near the entrance to an estuary or an inlet the ellipse may become practically rectilinear, while remote from such disturbing elements the ellipse will be less eccentric. On an open shore exposed to the direct advance of the tidal wave these currents are generally weak, and are unimportant; but on an irregular shore line, where the advance of the tidal wave is at varying angles

with the beach, and the shore line is broken by bays, estuaries, inlets, or straits, these currents may become great factors in erosion and transportation of material. In the open sea, the maximum velocity of tidal currents is found at high and at low water, while the slack or the turn of the tide occurs at mean sea level.

When a small basin or a narrow channel opens into the ocean, it fills when the tide rises and empties when it falls. A flood current prevails during the period of rising tide, and ebb currents during that of falling tide. The slack of both flood and ebb coincides very nearly with high and low water. In general, near the shore line the situation is one intermediate between the free propagation of the tide in the open sea and the condition of filling a basin of small capacity; the regimen of the tide is there more or less altered; the slackings and reversals of the currents are no longer effected at mean tide, but come nearer the time of high and low water; and the maximum velocity is near mean tide. The velocities depart from their theoretical values and in order to interpret observed phenomena, frequently it becomes necessary to have recourse to considerations based on the hydraulics of water flowing under the influence of a surface slope.

Tidal currents, therefore, must be distinguished from the purely hydraulic currents occurring when the accumulated water on the coast due to the tide runs through a narrow inlet or strait to a large bay or sound, which may or may not have also some other connection with the ocean.

Ocean currents once established are modified by local circumstances, notably by irregularities of the bottom and of the shores. They produce eddies, whirls, and counter-currents. If a current comes very close to the entrance into a bay it causes within the bay a current in the opposite direction. Eddies and counter-currents are also formed in the lee of islands.

A sharp reduction in depth determines perturbations in ocean currents, which, on account of the violent reaction from the bottom, may acquire great increase in velocity. The reversal of a current on the coast does not always occur at the same moment, for the entire depth of water, but usually at the bottom before it may be noticed at the surface.

Wind Currents.—The wind blowing on the surface of the water tends to drag the water along with it, which quickly gives rise to ripples and small waves, rapidly increasing to large ones as the wind's intensity increases. It has been found by observation* that the wind current on the surface will have about 1½ to 2% of the velocity of the wind and will be about 20° in direction to the right of the direction of the wind in the northern hemisphere.

The ocean currents due to winds, although usually feeble, may at times have a velocity comparable to that of the regular currents observed at sea in calm weather. It is quite evident, therefore, that strong winds blowing along the direction of the shore line may give rise to exceedingly swift longshore currents under favorable conditions. If the regular sea current and that due to the wind are in the same direction, they strengthen each other. If they are in contrary directions, they may destroy each other. The wind current may

* "Tides and Their Engineering Aspects," by G. T. Rude, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1123.

also reverse the sea current; but by reason of the fact that the action of the wind is on the surface, the current due to the wind is itself a surface current, while the sea currents usually have a great depth.

It follows that a diminishing of the general velocity of the current on the surface may require an increase in the bottom velocity in order to provide for the flow that is demanded by the predominant phenomenon, such as the tide. This reversal of the surface currents and the reinforcement of the bottom currents are frequently observed, particularly by fishermen, who find it a warning of an approaching change of weather. It also accounts for the fact that an offshore wind usually promotes the building up of the beach.

The accumulation of water produced in a bay by the wind may cause currents of opposite direction at the surface and at the bottom. The surface waters, driven by the wind, tend to heap up, in more remote parts of the bay, to a level above that of the sea at the entrance. They may, by the violence of the velocity which they have acquired under the impulse of the wind, heap up to a level even higher than that corresponding to their new equilibrium under the combined action of their momentum and of the horizontal force of the wind which drives them. To return to their normal height, they can no longer follow the slope of the surface of the water where the wind would be opposed to their return; they therefore follow the bottom. The water comes in on the top and returns on the bottom in the form of an undertow.

Into bays or lagoons connected with the sea by inlets the waters driven by the wind penetrate and accumulate. They then escape, when the wind has changed in intensity or direction. This produces in the inlets a powerful flushing effect, or scour, which is very beneficial in the maintenance of depth.

The winds, therefore, often set up littoral currents along the coast. Such a littoral current does not exist when the wind is normal to the beach, whether it comes from the land or from the sea. The current is produced all the stronger as the direction of the wind departs more from that normal direction. The movement of the littoral current is in general agreement with the direction of the wind. The current, therefore, transports the sand in the direction of the prevailing winds. While difficult to observe, the velocity of littoral currents is greater near shore than farther at sea.

There are other currents such as seiche currents, planetary currents, salinity currents, etc., but those given are believed to be the ones principally concerned in longshore drifting.

Formation of Inlets

Inlets in the sandy cordon along the sea coast known as the "offshore bar" are probably formed naturally in one of three ways. First, the recurvature of a growing sand-spit may be such in degree and direction as to bend it toward an adjacent shore line so as to surround progressively an enclosed area of water. The connection of this body of water with the sea will become smaller and smaller as the spit grows until the opening reaches such dimensions that the velocity of tidal flow becomes sufficient to erode the end of the spit as fast as it forms, or its growth ceases for other reasons.

Such an inlet is at present in course of formation at Assateague Anchorage on the eastern shore of Virginia (Fig. 1). Sixty years ago the area now being enclosed by a growing sand-spit was open sea. A sand-spit, called Fishing Point, began to grow seaward in a southerly direction from Assateague Island and after extending about 2 miles recurved in a westerly direction toward the north side of Chincoteague Inlet, partly enclosing an area of nearly 6 sq. miles. A further extension of Fishing Point for 1 mile will completely enclose the anchorage area, and convert it into a bay or sound similar to many others along the coast. It will continue, of course, to maintain a small opening or inlet to the sea, as long as the tidal flow into and out of the enclosed area is sufficient to maintain itself against the encroaching sand.

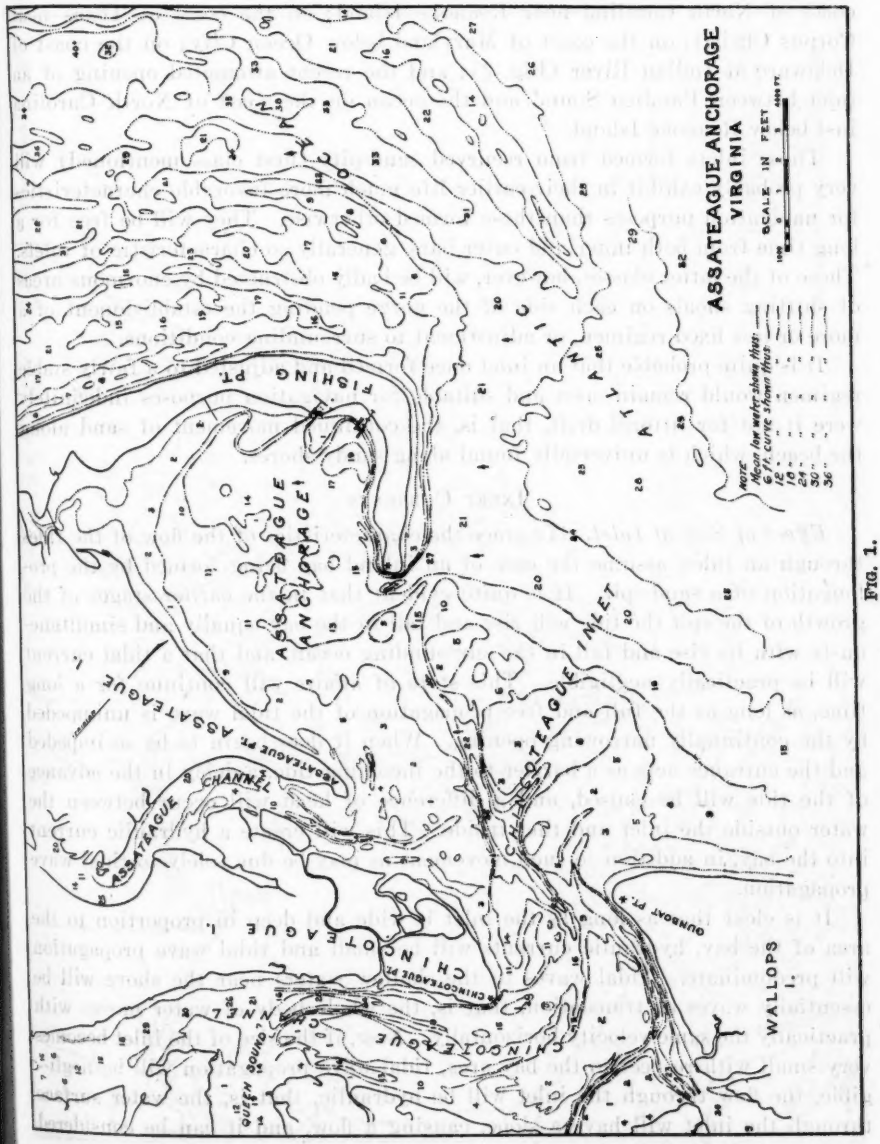
The neighboring Chincoteague Bay (Fig. 1) appears to have been formed in a similar manner by the extension of a sand-spit from the headland just south of Cape Henlopen. Indeed, the theory has been proposed that this is the normal method by which the barrier beach between bays or sounds and the ocean is formed.

This view is probably not correct, however, as will be explained in reference to the second, and in all probability the most usual, mode of original occurrence of an inlet. This is also intimately related to the mode of occurrence of the offshore bar, the theory of which is that a young shore line is just rising or emerging from the sea. While in such a case the smaller waves break on the beach, the larger and more powerful storm waves break farther out in shallow water where they cause a ridge of sand to form offshore. This gradually piles up until it finally emerges from the sea and forms the littoral barrier, or offshore bar, cutting off the water between it and the mainland to form the bay or lagoon.

This offshore bar does not emerge uniformly at the same rate at all points. Hence, although the major portion of the bar may be rising, a flow of water still maintains itself between the lagoon and the sea through the more or less numerous low points, which become more and more narrow until they are true inlets. As this offshore bar forms, longshore currents will start a movement of littoral drift along it, which will hasten the closure of those inlets having the weakest tidal flow and narrow those able to maintain the flow.

The third mode of formation of an inlet is for the barrier beach or offshore bar (separating the ocean from the coastal bays and sounds) to waste away by denudation, either by the waves, or by the wind, to a point where unusually high or storm tides overtop the beach and, flowing across it into the bay, erode a channel. The first two modes of formation are of infrequent occurrence and require much time; the third occurs often and may require only a few hours for its consummation. Some of the numerous recent examples along the coast of the occurrence of new inlets from breaches in the barrier beach are the new Beach Haven Inlet near Beach Haven, N. J. (Fig. 2), connecting Little Egg Bay with the ocean on the New Jersey coast, and Sinepuxent Inlet, connecting Sinepuxent Bay with the ocean on the Maryland coast.

There is a fourth mode of creating an inlet, that is, the intentional artificial opening of such by works of man. A number of attempts have been made to


ASSATEAGUE ANCHORAGE
VIRGINIA
SCALE IN FEET
Note
Mean low water shown thus:
6 Ft. curve shown thus
12 " " " " " "
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24 " " " " " "
30 " " " " " "
36 " " " " " "
ASSATEAGUE ANCHORAGE
FISHING PT.
ASSATEAGUE I.
ASSATEAGUE PT.
ASSATEAGUE CHANNEL
ASSATEAGUE INLET
CHINCOTEAGUE I.
CHINCOTEAGUE PT.
CHINCOTEAGUE CHANNEL
QUEEN SOUND
DUNBOAT PT.
WALLOPS I.

FIG. 1.

cause artificial openings at selected localities, but the usual history of such attempts has been failure, due to a lack of appreciation of the natural forces to be encountered and overcome and to failure to carry out the work on a sufficiently large scale. Instances of failure are attempts to open inlets on the coast of North Carolina near Roanoke Island; on the coast of Texas near Corpus Christi; on the coast of Maryland below Ocean City; on the coast of Delaware at Indian River (Fig. 3); and the recent attempted opening of an inlet between Pamlico Sound and the ocean on the coast of North Carolina just below Roanoke Island.

Those inlets formed from recurved sand-pits (first class mentioned) will very probably exhibit in their earlier life much more favorable characteristics for navigation purposes than those formed otherwise. They will be free for a long time from both inner and outer bars, generally so characteristic of inlets. Those of the latter classes, however, will be badly obstructed by enormous areas of shifting shoals on each side of the gorge pending the establishment of a more or less fixed regimen, or adjustment to surrounding conditions.

It is quite probable that an inlet once formed and adjusted to a fairly stable regimen would remain open and suitable for navigation purposes indefinitely were it not for littoral drift, that is, the continued movement of sand along the beach, which is universally found along sandy shores.

Inlet Currents

Effect of Size of Inlet.—To trace the characteristics of the flow of the tides through an inlet, assume the case of an inland bay being formed by the prolongation of a sand-spit. It is quite evident that in the earlier stages of the growth of the spit the tide will rise and fall in the bay equally and simultaneously with its rise and fall in the surrounding ocean, and that a tidal current will be practically negligible. This state of affairs will continue for a long time, as long as the full and free propagation of the tidal wave is unimpeded by the continually narrowing opening. When it does begin to be so impeded and the entrance acts as a barrier to the incoming tide, a delay in the advance of the tide will be caused, and a difference of head will occur between the water outside the inlet and that inside. This will create a hydraulic current into the bay, in addition to such movement as may be due solely to tidal wave propagation.

It is clear that as long as the inlet is wide and deep in proportion to the area of the bay, hydraulic currents will be small and tidal wave propagation will predominate. Tidal waves in the shallow waters near the shore will be essentially waves of translation, that is, the whole body of water moves with practically the same velocity horizontally. Now, if the size of the inlet becomes very small with respect to the bay area, tidal wave propagation will be negligible, the flow through the inlet will be hydraulic, that is, the water surface through the inlet will have a slope, causing a flow, and it can be considered mathematically in accordance with known hydraulic laws.

The ideal inlet thus formed would soon begin to deteriorate due to the material supplied by littoral drift taken in and deposited by the flood tide and so tend to become indistinguishable from the inlet formed by breaches of the beach and similarly obstructed by bars.

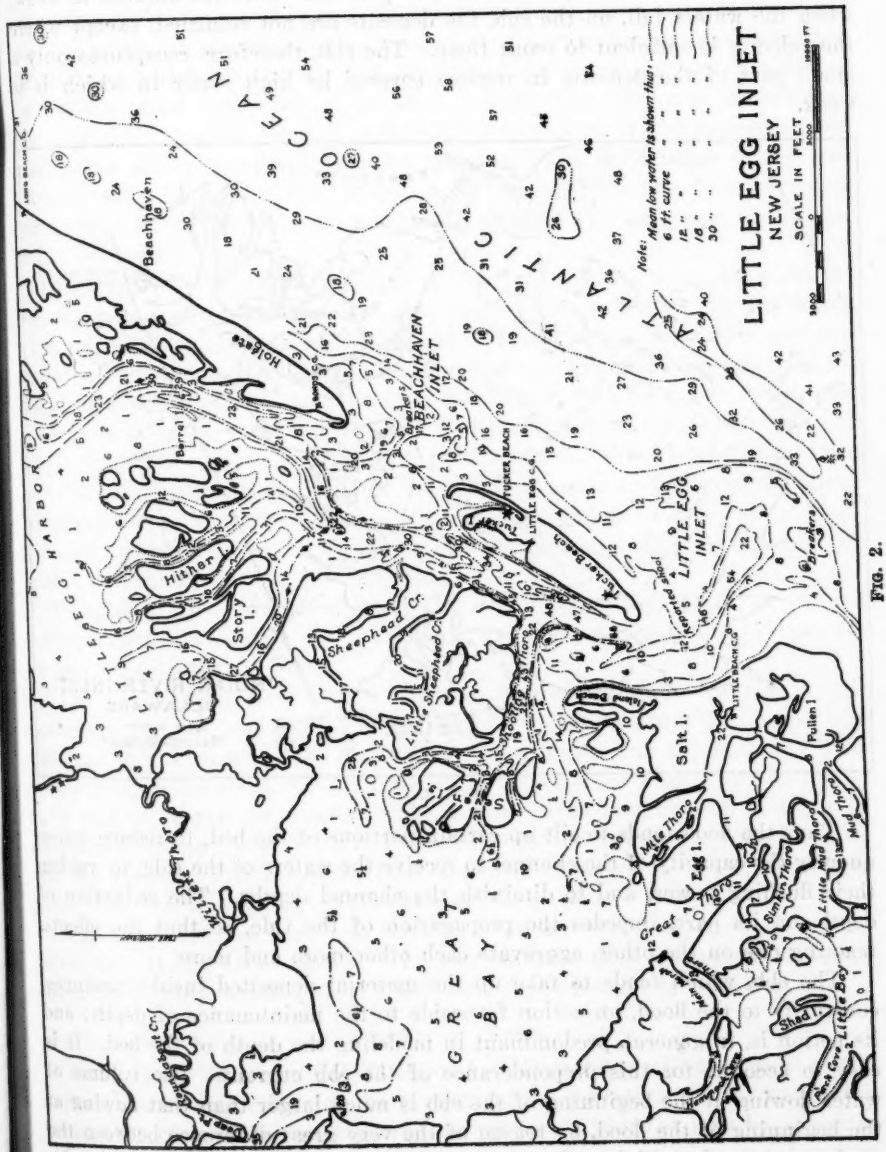
LITTLE EGG INLET
NEW JERSEY
SCALE IN FEET
Note: Mean low water is shown thus
6 ft. curve
12 " "
18 " "
30 " "
ATLANTIC OCEAN
BEACHHAVEN INLET
LITTLE EGG INLET
LITTLE EGG HARBOR
GREAT BAY
Beachhaven
Holgate
Higbee Beach
Tuckers Beach
Brigantine
Higbee I.
Story I.
Sheephead Cr.
Little Sheephead Cr.
Salt I.
Pullen I.
Mud Thorofare
Little Mud Thorofare
Egg I.
Great Thorofare
Shed I.
Little Boy
Reed Cove
Hampton Gut
Deep Pt.
Wilkinson Cr.
Wilkinson Thorofare
Mile Thoro.
Bonita Thoro.

FIG. 2.

Flood vs. Ebb Currents.—The currents of the flood and of the ebb have a different effect on the bottom from the point of view of depth maintenance. The flood current carries into the stream drift which is deposited throughout the entire bed, particularly in the convex portions where the current is weak; when the waters fall, on the ebb, the deposits are not returned, except where the velocity is sufficient to scour them. The ebb, therefore, recaptures only a small part of the deposits in regions covered by high water in which it is weak.

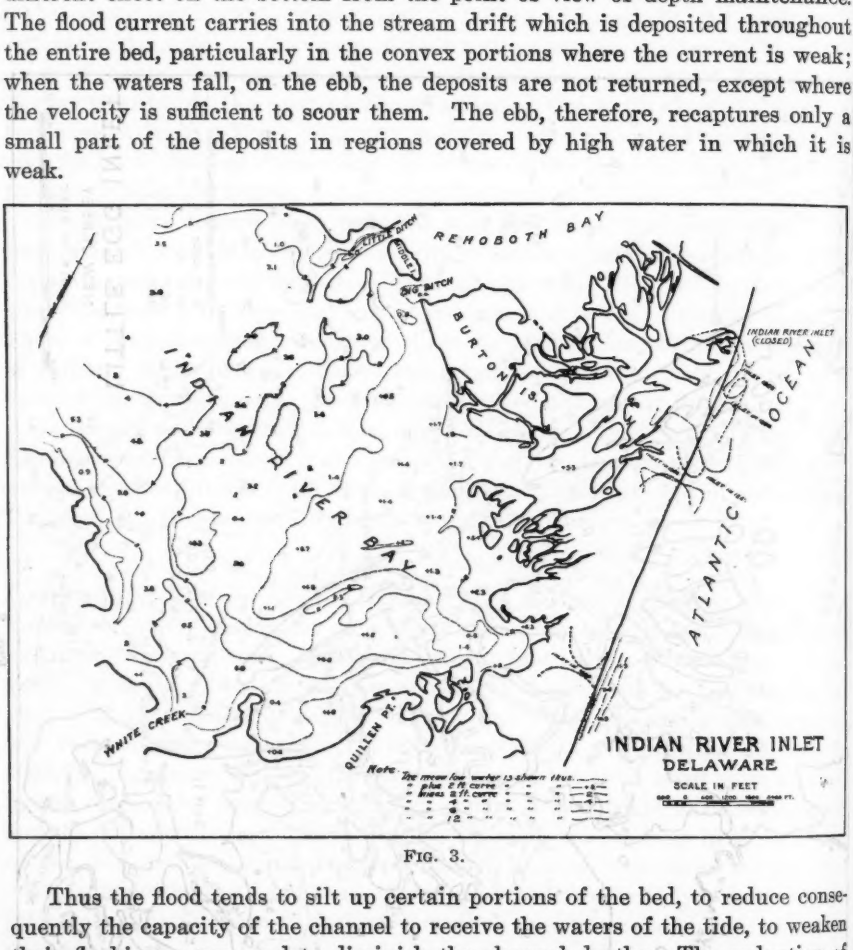

FIG. 3.

Thus the flood tends to silt up certain portions of the bed, to reduce consequently the capacity of the channel to receive the waters of the tide, to weaken their flushing power, and to diminish the channel depths. The reduction of depth, on its part, impedes the propagation of the tide, so that the effects, reacting one on the other, aggravate each other more and more.

The ebb, which tends to take up the material deposited inside, produces, contrarily to the flood, an action favorable to the maintenance of depth; and its action is, in general, predominant in modeling the depth of the bed. It is easy to account for this preponderance of the ebb current. The volume of water flowing at the beginning of the ebb is much larger than that flowing at the beginning of the flood, by reason of the very great difference between the surfaces covered at high and at low water; and the total volume of the ebb is greater than the total volume of the flood by the entire volume of any fresh-water discharge that may occur. On the other hand, it must be remembered that the flood tide often rises at a more rapid rate than the ebb falls, due to

the conformation of the channel bed being such as to impede the rate during the earlier stages, and that the flood operates on an increasing hydraulic radius while the ebb has a decreasing hydraulic radius.

The predominant tendency of the flood tide to take in all available drift manifests itself in several ways. It has been found from current measurements that the surface velocities in tidal channels are generally greater for the ebb tide; at mid-depth the ebb and flood velocities are about equal; while at the bottom, flood velocities are greater, making the flood a more effective scouring agency. The ebb is also diluted to a greater or less extent by any fresh-water discharge that it may be carrying. The flood being saltier and heavier, will push in beneath the ebb, thus rendering the bottom water slack during a portion of the ebb, or else actually causing, during a part of the ebb, an inward transportation.

Quite often the maximum velocity of the flood tide is greater than that of the ebb, giving it a preponderance of transportation capacity. This is found when the entrance of the flood tide is held back and delayed by shoals, or other causes which change the form of the tidal wave, the front of the wave becoming steeper than the back, and the flood tide stronger. Under such conditions, the strong flood currents may be able to transport more and larger drift or débris than the weaker ebb currents can remove. Certain observers have reported also that, while during the ebb tide the bottom and sides of the channel appear to be hard and well consolidated, they appear to be looser and less compact on the flood, which results in their being more easily eroded and the eroded material being put in suspension.

In spite of the harmful effects which the flood current produces in promoting deposits, its action also contributes in a certain measure to the maintenance of the depths.

The young flood ascending the bottom of the bed while the ebb is still descending on the surface has been likened to a plowshare which, under the pressure of the water which loads it down, may cut out the bottom. This young flood makes itself felt naturally in the deepest portions of the channels; it is said to "carve the passes". The bottom particles taken up and put in suspension by the young flood, and carried in by the waters, are recaptured by the ebb current which carries them seaward. The young flood, therefore, assists the ebb in digging and deepening the passes.

The actions of the flood and the ebb, however, do not always make themselves felt at the same places. The two currents may indeed cut different channels, between which is dead water, very favorable to the deposit of silt.

Deposit of Drift

Conditions Favorable to Drift Deposit.—All currents will accumulate drift and sediment, depending on their velocity, the agitation of the water, the availability of erodible material, and the ease with which it may be placed in suspension. They will deposit their burden when the velocity is reduced to the proper degree and the water remains comparatively quiet for a greater or less length of time; also, when a current is already near saturation with a load of drift and a greater burden is added to it, it will deposit the excess. Tidal

currents, and hydraulic currents having the same periodicity as tidal currents, will usually deposit mainly on the slack of the flood tide, but in sheltered places also on the slack of the ebb.

When the longshore drift has been transported by the various currents to a point of deposition, and the process has continued for a considerable length of time, a state of equilibrium may finally be reached where successive deposits have so reduced the area available for such that the currents become swifter, and are able to remove further deposits as fast as made. Thenceforth, as long as the velocity of the currents remains unimpaired, no further shoaling will occur, and all drift brought in by the flood tide will be swept out by the ebb and returned to the sea.

In case fresh water discharges into a bay receiving a tidal flow, the scouring effect of the ebb will be correspondingly greater than the filling effect of the flood, and the bay may be expected to keep reasonably clear of deposits. In case, however, the bay receives little or no fresh-water flow, the flood will generally predominate over the ebb, and deposition will exceed scour until the channels are filled and even the tide is excluded from the basin.

Since drift carried by currents which are periodic, is also periodic in its movement, its path is far from being a simple one. It is subject to longer or shorter halts when deposited, and to being carried back and forth many times over the same ground. Being subject in these journeyings to constant attrition, the particle becomes smaller and smaller, and proportionately more easy to transport. The particle tends eventually to reach the sea, since, due to fresh-water discharge or to the effect of other currents, it will gradually work its way outward with the ebb.

In general, it may be stated that all shore drift, whether beach drift or longshore drift transported by any type of current, when near an inlet or estuary, will be gathered up by the flood tide and transported into the bay, where it will be deposited, and from which it may or may not in turn be scoured out and returned to the sea by the ebb. This process may be repeated as many times as the particle is transported within reach of the currents at any inlet.

Mud Deposits.—Mud represents the ultimate residue of the grinding of the materials of every kind under the ceaseless action of these forces. In agitated water, it remains in suspension indefinitely. It is deposited slowly in calm water and does not acquire any cohesion until after a time of rest, the duration of which varies according as the mud is of a sandy or a clayey nature. Freshly deposited mud can often be pumped as a liquid, but little by little it takes on coherence and then behaves in the manner of a damp clay.

Mud is deposited wherever the water is sufficiently calm. Muddy bottoms at sea, consequently, reveal the depth to which the waves are no longer felt ordinarily, calm being one of the conditions prevailing in bays and lagoons. Practically all of these, therefore, require works for mud removal, at least in some of their parts, to prevent ultimate deterioration.

Inlets are most often an abundant source of mud, because they are vast grinding mills where the particles of drift are alternately carried up by the flood and down by the ebb.

The mud in suspension in the waters of the ebb flowing from an inlet may be carried along the beach by the prevailing littoral currents in the sea. It is thus transported for a certain distance from the mouth to another part of the coast. When the ebb current stops in this new region to which the mud has come, the young flood again carries the waters landward and into the inlets and bays, debouching there. The filling is done with the already turbid waters which there again deposit a greater or less quantity of mud.

When deposited by flood tides in basins, mud consolidates more rapidly than sand, particularly deposits of an argillaceous nature. The still water promotes a rapid accumulation of marine growth, the roots of which further tend to consolidate and bind the mud into a compact mass not easily eroded or put into suspension in water. Sand deposits do not take on this consolidation except where thoroughly permeated with mud. It is found in excavating artificial channels in marine marshes that for those cut in the consolidated mud, where marsh grasses are abundant, the channels are of a more permanent nature, have firmer and more satisfactory banks, and are less liable to shoaling than those cut in soft mud, pure sand, or sand not containing enough mud to support marine growth, or to enable consolidation to occur. Channels cut through such materials are extremely unstable, the bank flattening out quickly, and the channel rapidly widening and shoaling by a breaking down of the banks. It follows that, whenever practicable, artificial channels to inlets should avoid a route through unstable and shifting sand-bars, and lie as far as practicable in consolidated marshes.

Channels of the latter type will not erode and increase in size under an increased current velocity as do those in sandy material, but they soon become adjusted to local tidal conditions, and serve navigation better.

Migration of Inlets

Causes.—Many inlets are subject to a lateral displacement, termed migration. This may be due to any cause affecting the physical condition of the two sides of the inlet unequally. The most frequently assigned cause is an excess of beach drift accumulating on one side, thus forcing the channel against the other side with consequent excessive erosion. By such a growth, one side may extend outward and in front of the other, so as completely to overlap it, forcing the outer bar a considerable distance along the shore from the normal location in front of the gorge.

Sometimes, if conditions permit, particularly if the inner delta be not symmetrical with respect to the axis of the gorge channel, or if the area of the lagoon be not symmetrical with it, the gorge will keep pace with the outer bar in its movement, and the whole inlet will migrate; while at other times only the outer bar migrates, the gorge remaining in the same place. This results in a lengthening of the inlet until it may become simply a long narrow channel approximately parallel to the beach line, connecting a new outer bar with the bay. This is most liable to occur when the supply of beach drift is plentiful.

Under other conditions, the moving outer bar may recover its original condition, going through a regular cycle of moving in one direction until it lies

close to the beach, then breaking out on the opposite side, and again swinging in the same direction as before. At times, while one side of the inlet is maintaining itself fairly stably, the other side will start to recede, due to excessive erosion or lack of normal supply of littoral drift, and a widening of the inlet will ensue. In this case a middle ground shoal will eventually form, and the inlet may split into two parts, one of which will remain in the normal location, while the other will migrate in the direction of the original widening. Such a branch inlet usually is short lived.

Direction.—The migratory effect of littoral drift on an inlet appears to depend on the relation between the respective volumes of drift and of tidal flow. If the volume of drift is small with respect to the tidal flow it will have little effect, as in that case the ratio between volume of drift and volume of water is small and the scouring capacity is correspondingly large. If the volume of drift is large, its effect will become appreciable, and will increase with an increase of the ratio of volume of drift to volume of water.

The extent to which migration due to littoral drift may extend is determined by the extent of the connecting bay or lagoon and the size of the connecting thoroughfare. As soon as the distance between the ocean and the bay by way of the inlet and thoroughfare becomes too great, causing undue hydraulic resistances, the inlet begins to close, and another inlet more favorably situated may form, or the bay may remain for an indefinite period effectively separated from the ocean, the excess fresh-water accumulation finding its way to sea by seepage through the unbroken sand barrier. Indian River Inlet on the coast of Delaware (Fig. 3) closed in such a manner and that large body of water remains separated from the ocean except by the long artificial Delaware Bay-Rehoboth Bay Canal. On the coast of New Jersey, Little Egg Inlet (Fig. 2) migrated until its connection with Little Egg Bay was long, when a new and more favorably located inlet formed near Beach Haven, which has resulted in great deterioration of Little Egg Inlet, and its future closure seems possible.

Occasionally, the direction of migration of an inlet is opposite to that of beach drift. It is very apparent, for instance, that there is an unusually heavy movement of beach drift southward from the headland at and south of Cape Henlopen to Assateague Island. This movement goes directly past Indian River Bay, Delaware, and under the foregoing conception of the effects of such movement an inlet connecting Indian River Bay and the ocean should migrate south.

The original survey, made in 1847, however, shows that the Indian River Inlet (Fig. 3), recently closed, was at that time farther south, and that in previous times it had been located still farther south than then shown, the total distance of northerly migration since 1847 having been 2 miles. The predominant direction of the longshore current on this part of the coast is not known accurately, but the best evidence obtainable is that it is southward.

It is believed that this counter-drift migration may possibly be accounted for as follows: The southerly currents are here carrying longshore drift in such quantity as to be at or near the point of saturation for the existent velocity. Any increase or decrease of that velocity would result in a change

of the saturation point, giving the current a greater carrying power when increased, and causing a deposit of the excess burden when the velocity is decreased. The same is true of the ebb flow out of the inlet. When these two currents meet (Fig. 4), approximately at right angles during the ebb at inlets, the resulting current is in the direction of the resultant of the two forces, and its velocity is less than the sum of the two velocities. Nevertheless, it carries the sum of the burdens of drift of the two currents. The new current, tending down the coast, at an angle is thus supersaturated with silt, and deposits its excess on the south side, thus tending to build up the south shore at a greater rate than erosion occurs. This constant accession of an excess of sand at certain times appears to result in this apparently exceptional case of migration counter to the drift.

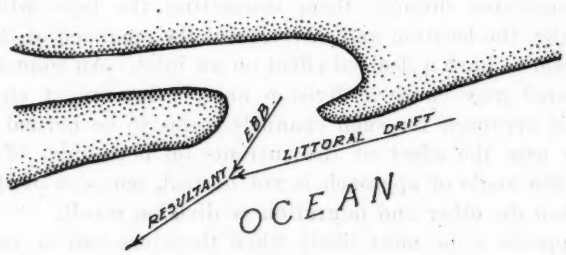

FIG. 4.

It was noted, however, in this case that the channel through the inner bar approached the inlet from a southerly direction, then impinging on the northerly side of the inlet it was diverted southerly again, and by a second diversion crossed the bar in a northeasterly direction. It is apparent that, at the first point of deflection, erosion would occur to a degree that would cause a continued northward movement of the bend, and that the bend would increase to a point where the narrow beach between the thoroughfare and the ocean would be breached, causing a new inlet to break out northward of its old position. The southerly movement of littoral drift would then extend the northerly side of the inlet in such a manner as to restore previous conditions, but at a more northerly location.

Assuming that the southerly flow of drift at an inlet is constant and at or near the saturation point for the velocity, the stream of drift will be turned into the inlet by the flood tide, practically none of it passing and going on to the southward. When the tide turns, the ebb takes up most of the load brought in by the flood tide and goes out with it, meeting the oncoming outside stream. On the southerly side of the inlet the continuity of the alongshore movement, therefore, will be intermittent, the deficient material having been deposited as shown.

Another instance of migration of an inlet in a direction opposite to the movement of littoral drift is at Aransas Pass on the coast of Texas. There the inlet or pass for many years persistently moved in a southerly direction while the resultant wind and consequent drift movement is northerly. This was due to the fact that the principal thoroughfare from the inner bay flows

in a southerly direction parallel to the coast line for a considerable distance, until it strikes normally against Mustang Island, where it turns a right angle and flows to sea. Normally, the tides in these bays are small and generate mild currents, but in times of "northers" the bay waters are violently driven south, and escaping through the Pass formerly caused erosion of Mustang Island, and a southerly movement of the inlet. This erosion and migration have now been stopped by the revetment of the bank on Mustang Island where the erosion was greatest and by the fixation of the Pass by means of jetties.

Division of Inlets

Where the inner bars of inlets have formed large marshes with long channels or thoroughfares through them connecting the bays with the ocean through the inlet, the location and direction of the approach of these channels to the inlet seem to have a decided effect on an inlet. An improperly located approach channel may cause a division or splitting up of an inlet. The proper angle of approach for such channels seems to be normal to the coast line, in which case the effect of the currents on both sides of the inlet is the same. If the angle of approach is not normal, one side of the inlet may erode faster than the other and migration or division result.

Division appears to be most likely when there are two or more channels converging at the inlet, one suitably located or normal to the coast line, and the others making an acute angle with it. In such a case a regular cycle of division often results. The discharge from a channel having its approach at an acute angle will cling close to its side of the inlet, and will induce excessive erosion and consequent widening on that side with the other results already described. The off-springing inlet generally closes when it has migrated too far from the main stem to maintain itself, and the cycle begins anew.

This condition prevails at Great Egg Inlet and at Absecon Inlet, New Jersey, where the cycles described can be traced. Both inlets are now in the early stages of a cycle, the north shore in each case being in process of erosion. In the case of Great Egg Inlet many blocks of the Borough of Longport have disappeared. A determined effort is being made to arrest the erosion, but without attacking the basic cause, the erosive effect of Risley Channel flowing to sea close around this point.

At Absecon Inlet (Fig. 5) the channel called Main Channel, by taking a short cut to sea, had widened the gorge, and caused an extensive, troublesome, middle-ground shoal, and had washed away many acres of Brigantine Island. In connection with the construction of a road to Brigantine Island this channel has now been permanently closed, and the restoration of the southern end of the Island is now in progress, with an accompanying betterment of the outer bar at Absecon Inlet.

Critical Velocity

If the velocity becomes greater than a certain critical value for the material composing the banks and bed, erosion will occur until the velocity

ABSECON INLET
NEW JERSEY
SCALE IN FEET
ATLANTIC OCEAN
ABSECON INLET
ATLANTIC CITY
BRIGANTINE INLET
Brigantine Channel
Mud Thorofare
Brigantine Shoal
Grassy Bay
Reed Bay
Absecon Bay
Great Thorofare
Lake Bay
Breakers
Note. Mean low water is shown thus

FIG. 5.

is reduced to the critical point by an enlargement of cross-section, or by an increase of friction due to an extension of the inner and outer bars. If the velocity falls below the critical value, the current will be too weak to scour out the detritus brought in by the flood tide, and shoaling will occur. It is quite evident, therefore, that any lessening of the velocity of the currents through an inlet is a matter of some importance.

Since most inlets in the course of time have become automatically adjusted to the critical velocity, which for inlets on sandy shores will be quite uniform for different localities, it can be determined by actual measurement of several inlets having more or less the same character of bed or channel. The critical velocity will be found in the narrow portion of the inlet called the gorge, or throat, and is the mean velocity found there at the strength of the flood or ebb tide. Where the inlet has widened out and has a greater cross-sectional area than in the gorge, the velocity will be less than the critical and the liability of shoaling correspondingly greater.

Shifting of Navigable Pass

Direction of Change.—With but few exceptions, the outer bars of all inlets on the Atlantic Coast undergo an identical cycle of change in location of the pass or place of greatest depth. It seems to make little difference from which direction the predominant littoral drift comes, the point of greatest depth on the bars moves at a slow rate from a central location to the right in a southerly or southwesterly direction until the outer channel is bent around nearly parallel to the coast line, and the bar is close inshore. As the outer channel approaches this position, considerable erosion occurs on the adjacent beach, particularly on the outer edge of the lip of the inlet. This is sometimes considered a migration of the inlet, but is not such in reality, as the eroded beach will again build up in a later part of the cycle, and the gorge remains in the same location during the entire period. When the bar has thus reached a point close in to the shore, its rearward and higher portion or base is more and more subject to the tendency of the ebb tide to flow to sea straight across the shoal and is exposed to the attack of northeasterly storms, which eventually work a passage through that enlarges itself enough to become again the principal one. The old channel closes and the cycle begins anew.

Cause.—This action has been the subject of many attempts at explanation. The one most generally given is that this bar passage is pushed over farther and farther by the encroachment of littoral drift, that is, the direction of movement of the bar is that of the predominating drift. This explanation, however, is not satisfactory, since the prevailing direction of littoral drift often varies with the season of the year and varies with the locality, while the direction of swing of the bar does not vary with the seasons. On the Atlantic Coast it is always in the same direction regardless of locality, yet the rate of swing does vary, being greatest in winter and least in summer.

The predominant direction of movement of littoral drift is clearly that of the predominant direction of wind movement. In winter the northeast and east winds prevail on the Atlantic Coast in point of length of time and

in intensity, and cause a southerly drift. In summer the southerly and southwesterly winds prevail, and cause a northerly drift. Since the change of direction of the littoral drift does not change the direction of movement of the bar, some other cause must be sought.

It is well known that storm waves driven to the shore break in a depth of water approximately equal to their height, and that in breaking they tend to place the bottom material in suspension and to hurl it toward the shore with the foaming water. Such waves breaking against a bar tend to push the bar back by thus eroding the point on which they break and to heap some of the material back on the bar itself. At the same time some of this material is apparently pushed sideways, so that the face of a bar on which heavy storm waves have beaten for some time appears flattened and elongated in plan.

The northeasterly and easterly storms, being the most prevalent heavy storms striking the Atlantic Coast, will strike the outer bar on its northeasterly point, flattening or pushing it back to some extent and elongating it. Since elongation cannot take place to any considerable extent northward due to the nearness of the shore line, it must take place southerly or southwesterly, the elongation extending into, and encroaching on, the channel, thus pushing the bar passage farther and farther in that direction. This constitutes the change of location or shifting observed.

Another effect of the strong northeasterly and easterly winds also contributes to this movement of the deepest water—the friction of the wind blowing on the surface. This friction causes an ocean current to set shoreward, the velocity of which will be about one-fiftieth the velocity of the wind. On the ebb tide, the currents flowing from inlets generally in a southeasterly direction due to the trend of the coast line will meet this outside wind current, and the resultant current will then set southerly or southwesterly, materially affecting the shoals on that side of the channel and promoting their erosion and the swing of the channel in that direction. When, due to these storm attacks, great erosion of the bar shoal occurs on the northeasterly side and the bar is there lowered to the degree that the low spot again becomes the principal channel, the channel is said to "break out" to the northeast.

This theory of the movement of the point of greatest depth on ocean bars has been deduced from a long series of observations made on Absecon Inlet, New Jersey (Fig. 5). In this case it can be asserted with great confidence that the shifting of the point of greatest depth on the bar southwesterly is due almost entirely to that cause, the direction of the littoral drift having no observable effect on the matter of shifting. A single northeaster has been observed to push as much as 100 000 cu. yd. of sand in a single day into the channel on the outer bar, by the elongation of the northeast shoal, resulting in a decrease in depth on the center line of the channel by 6 to 7 ft.

These excessive changes are observed only during and after heavy northeasters or easters. Subsequent to such a sudden change a gradual improvement in depth and an erosion of the elongated portion of shoal will be noted until the next northeaster, when a still further encroachment will be made.

A conclusion to be drawn from these facts is that if, by means of jetties or the natural trend of the shore line, the inlet should be so faced that the outgoing tide flowed directly into the prevailing storm direction, the bar, being free from this effect, would have a more permanent location and would maintain itself better.

Erosion

The currents flowing back and forth through the inlet have a certain capacity for eroding the bed and banks due to the velocity of the water. This eroding capacity is supposed to vary with the square of the velocity. Since, however, a deeper channel has a greater weight of water in it, and since depth tends to the formation of eddies and boils, which also tend to increase the amount of material in suspension, eroding capacity is greatly dependent on depth. Shallow channels are, in general, quite free of eddies and boils. It has been stated* that, when the discharge is constant so that the velocity changes with the slope, the capacity of a stream to carry a burden varies with the fourth power of the velocity, considering the mean velocity instead of the bed velocity. From this it may be seen that a small change in velocity must greatly affect the carrying capacity of the stream and induce deposit or scour.

The working force by which a channel is cut and kept open is the energy contained in the flowing water. This energy, as with all energy efficiently applied to useful work, must be concentrated into the most compact form possible for application to the work. The amount of kinetic energy available in any body of moving water is measured by one-half the product of its mass and the square of its velocity $\left(\frac{1}{2} m v^2\right)$. As already shown the velocity of water in an inlet at the strength of the tide will seldom vary much from a constant critical value. To increase the available working energy it is necessary to increase the quantity of water flowing and prevent the waste of its energy by dissipation of its mass due to lack of concentration. That means that the water should be kept in narrow, deep channels instead of in wide channels of equal carrying capacity.

This can be shown mathematically from the hydraulic equation, $r = \frac{a}{p}$, r being the mean hydraulic radius, and a and p being the channel area and wetted perimeter, respectively. This equation transformed into the form, $a = r\,p$, means that a is equal to an imaginary area in which r is the mean depth and p the mean width. Since r varies inversely with p, for a constant area, and a large value of r is desired for high velocities, p must be kept as small as possible, that is, the ratio, $\frac{p}{r}$, should be a minimum. The shortest line that can enclose a given area is the arc of a circle, hence the smallest value possible for p for a given value of a can be found as follows:

* *Professional Paper No. 86*, U. S. Geological Survey.

Let R = the radius of the circle the area of which is $2a$, then,

$$a = \frac{\pi R^2}{2}$$

and,

$$p = \pi R$$

therefore,

$$r = \frac{\pi R^2}{2 \pi R} = \frac{R}{2}$$

and the ratio,

$$\frac{p}{r} = 2 \pi = 6.2832 +$$

which is the smallest possible value of the ratio, $\frac{p}{r}$. In general, it will be much greater since, in Nature, no stream even approximates having a mid-channel depth nearly equal to one-half its width, as the banks usually will not stand at the required slopes. It is believed that a practicable working value for this ratio should lie between 20 and 30.

Relation Between Inlet and Lagoon

The regimen and capacity of inlets and their respective lagoons are interdependent, and these bodies of water are closely related in many ways. For example, the form and capacity of the inlet will determine the volume of the tidal prism within the lagoon, and the form, area, and location of the lagoon will determine whether it is possible by utilizing the tidal flow to and from this lagoon to scour out a navigable channel through the inlet. The size of the body of inland water is thus a factor in establishing the width and, to some extent, the depth in the gorge through which waters enter and leave the bay as the tide outside rises and falls.

Where the inland waters are extensive the position of the bay with reference to the axis of the inlet may be of importance. The tidal wave approaches the Atlantic Coast from a direction at right angles to the line of the beach, so it is fair to assume that the theoretical normal position of the axis of the currents through the inlet, if undisturbed by forces tending to modify it, should be through the middle of the inlet in a direction at right angles to the line of the beach.

Also, if sand is being carried by the longshore currents past the mouth of the inlet and some of it be seized upon by these hydraulic currents, the flood will carry it into the bay, depositing most of it just beyond the end of the gorge, where the currents are dissipated in the open waters of the bay. Similarly, the ebb current will tend to carry material seaward a short distance and there deposit it. The result is to form a more or less symmetrical "delta" at each end of the inlet.

If an exact balance were maintained between the longshore forces approaching the inlet from opposite directions, these deltas might be symmetrical with respect to the current axis, while, if there is a preponderance in the forces from one direction, that symmetry probably would be disturbed.

A lack of symmetry to most interior deltas is probably traceable to the existence of an extensive body of open water on one side of the inlet with only a limited water area on the other. This inequality may result in building a preponderating bay shoal inside the inlet which would force the main channel to one side as this shoal grows.

Formulas Applicable to Flow Through Inlets

Statement of the Problem.—It is quite evident that a condition favorable to the navigability of an inlet requires a sufficient volume of water at the critical velocity to establish and to maintain a channel of the required dimensions. The ability to secure a flow of the requisite volume of water for this purpose will depend on the physical characteristics of the inlet and its bay. The height of tidal rise in the sea, the length of the channel connecting the sea and the bay, and the size of the bay, are the principal physical factors thus concerned, that is, they are the principal factors which contribute to the frictional resistances opposed to the flow of the water. An inlet of a certain size is capable under the influence of the head due to ocean tides of carrying a certain volume of water between the ocean and the bay. This volume poured into and out of the bay will cause its surface to fluctuate above and below a mean level, and the volume that flows in and out of the inlet under the tidal head must equal the volume flowing into and out of the bay. These volumes, if they can be determined independently, in terms of the constant physical elements of any inlet, and equated to each other, will give an equation expressing the relationship which must at all times exist between those constants, from which one unknown constant can be obtained when the others are known, and from which the effect of any change in physical characteristics on other characteristics can be predicted.

Formulas will be deduced to express this mathematical relationship. The hypotheses on which the theory of flow of water into a closed basin through a single inlet is based, are that the flow is in accordance with the usual hydraulic laws of water flowing in an open channel of uniform cross-section under the influence of a surface slope; that the surface slope will vary with the levels of the respective surfaces of sea and basin; that the sea level varies with the ocean tides; and that the basin level varies with the quantity of water flowing into or out of it under the influence of that tide. The area of the inlet channel is assumed as constant, as it would complicate the solution too greatly to allow for its variation with the stage of the tide. Its value at mean tide is used.

While not strictly in accordance with observed facts, the variations of both surfaces are assumed to be sinusoidal, that is, to vary according to a sine law. These variations will not coincide in phase, since one is due to the effects of the other, and a certain lapse of time, which is constant for given conditions, is found between the occurrence of similar stages of the variations. This is termed the "lag".

Starting at mean sea level on a rising tide, the order of occurrence of the various phenomena will be as follows: At that time the level of the basin surface will be below that of the sea, and its rate of change of level will be

less than that of the sea, which will be at its maximum. The slope will be positive, or from the sea to the basin, and the current, or tide as it will be called for simplicity, will be in flood. The flood will continue to run at a diminishing rate, since the rate of increase of basin level now becomes greater than the rate of increase of sea level, which will reach its maximum height and begin to fall while the flood is still running, but at a decreasing rate.

At a time after high tide equal to the time of lag of the basin tide, the sea level will have fallen somewhat until it is exactly equal to the basin level, the surface slope will be zero or level; the current will be slack, neglecting all velocity due to inertia; and the basin will stop rising, that is, it will have reached its maximum height. Since there is no more water being admitted into it, it is manifestly impossible for it to rise further. Immediately as the sea tide begins to fall below the basin level, a negative slope is established, and the basin begins to empty its contents into the ocean by an ebb tide or current. This increases in rate, since the ocean tide falls faster than the basin tide, until below mean sea level, when the rate of fall again decreases until the ocean tide has reached mean low water, and has risen again sufficiently to cause a zero slope on low water and, consequently, an ebb slack of the tide, which immediately changes to a positive slope and flood tide.

It is evident that this condition is quite different from the case where a canal connects two tidal seas. In that case the lag or difference in phase of the tides at the two ends of the canal is not due to the quantity of water passed through the channel into a basin of limited capacity, but is independent of the quantity so discharged and is due solely to the difference of time for the tidal wave to travel by different routes to the two ends. In the case under consideration the basin tide is not a true tide, but is a filling and emptying of a basin of limited capacity by the water flowing from and to the ocean under the influence of its tide. In the former case the heights to which the tide rises at the ends of a canal are in no wise dependent on each other, nor is the lag dependent on either of them; but in the present case the height to which the basin tide rises and the lag are definitely and mutually related to the height of the ocean tide.

Solution of the Problem.—In Fig. 6, let S be the sea in which the tide freely rises and falls, flowing through I, the inlet, or inlet and thoroughfare combined, into and out of the basin, B.

Let a = cross-sectional area of the inlet assumed as constant.
L = distance from S to B, or length of inlet, I.
p = wetted perimeter of I.
s = slope of surface of water in I.
r = hydraulic radius of I.
H = mean tidal variation in the sea.
h = mean tidal variation in Basin B.
v = instantaneous velocity of flow in I.
C = hydraulic constant.
q = instantaneous tidal flow of I.

Q = total tidal flow of I (total tidal prism).
t = time reckoned from mean rising tide.
T = total time of one tide.
l = difference in tidal phase at S and B, or lag.
m = area of B.

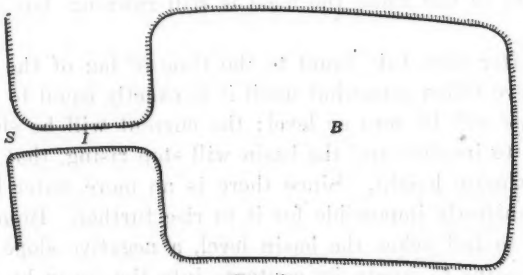

FIG. 6.

From hydraulics, the following equations are assumed:

$$v = C\sqrt{r\,s} \quad \ldots\ldots (a)$$

$$r = \frac{a}{p} \quad \ldots\ldots (b)$$

$$d\,q = a\,v\,d\,t \quad \ldots\ldots (c)$$

In Fig. 7 let the sine curve,

$$Y = \frac{H}{2}\sin 2\,\pi\,\frac{t}{T} \quad \ldots\ldots (1)$$

represent the tidal curve of the sea, and the curve,

$$y = \frac{h}{2}\sin 2\,\pi\left(\frac{t-l}{T}\right) \quad \ldots\ldots (2)$$

represent the tidal curve, assumed as sinusoidal, in the basin. The dotted line, also sinusoidal, is the algebraic difference between these two curves, and it

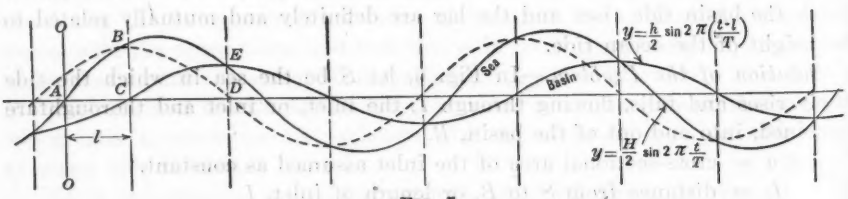

FIG. 7.

will represent the fall, or head, existing between the sea, and the basin at any time, t. Since this fall or head divided by the distance between the sea and the basin gives the mean slope of the water surface, this dotted line is proportional to the slope and its equation may be written,

$$s = \frac{1}{2\,L}\left(H\sin 2\,\pi\,\frac{t}{T} - h\sin 2\,\pi\left(\frac{t-l}{T}\right)\right) \quad \ldots\ldots (3)$$

From Fig. 7, it is manifest that E, the point where the curves, Y and y, intersect, is at the highest point of the curve, y, and that the ordinate, $E\ D$, is equal to $\frac{1}{2}\ h$. It is also clear that $A\ C$ is equal to the lag, l, and that $C\ D$ is equal to one-fourth the tidal period, or $\frac{1}{4}\ T$. Hence, the particular value for t at the point, E, is,

$$t = l + \frac{T}{4}$$

Equating the two values of $E\ D$,

$$\frac{1}{2}\ h = \frac{1}{2}\ H \sin 2\ \pi \left(\frac{l + \frac{T}{4}}{T}\right)$$

or,

$$h = H \cos 2\ \pi \frac{l}{T} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

which is the fundamental equation expressing the relationship between H, h, and l. It shows that as h increases to a value near H, the lag becomes smaller and the curve, y, approaches coincidence with Y, the slopes becoming smaller and the velocities negligible. This is the case when the basin is too small to receive the flow the inlet is capable of discharging. As h decreases, the angle of lag increases, and it is apparent that should h become zero the lag would be 90°, the curve, y, would disappear, and the slopes would reach the maximum possible values at high and low tide. This occurs when the basin is relatively very large, so that the inflowing water has no appreciable effect on its level, and is the most favorable condition for high velocities through the inlet.

Expanding Equation (3) and substituting for $\cos 2\ \pi \frac{l}{T}$ and $\sin 2\ \pi \frac{l}{T}$ their respective values, $\frac{h}{H}$ and $\frac{1}{H} \sqrt{H^2 - h^2}$, derived from Equation (4), Equation (3) may be written,

$$2\ L\ s = \frac{1}{H}\ (H^2 - h^2) \sin 2\ \pi \frac{t}{T} + \frac{h}{H} \sqrt{H^2 - h^2} \cos 2\ \pi \frac{t}{T} \ldots\ldots (4a)$$

All sinusoidal curves may be expressed in the general form, $y = A \sin (B\ x + K)$, A, B, and K having any values. A is termed the amplitude factor, B, the periodic factor, and K, the phase factor. In the case at hand, the value of B will be unity, as the desired curve will have the same periodicity as before, but proper values must be found for A and K.

Placing the general expression equal to $2\ L\ s$,

$$2\ L\ s = A \sin (B\ x + K) = \frac{1}{H}\ (H^2 - h^2) \sin 2\ \pi \frac{t}{T}$$

$$+ \frac{h}{H} \sqrt{H^2 - h^2} \cos 2\ \pi \frac{t}{T} \ldots\ldots\ldots\ldots\ldots\ldots (4b)$$

In Equation (4*b*), x will be replaced by $2\pi\frac{t}{T}$. The value of s is zero when $t = l + \frac{T}{4}$. Making these substitutions, Equation (4*b*) becomes,

$$A \sin\left[2\pi\left(\frac{l+\frac{T}{4}}{T}\right) + K\right] = 0$$

whence,

$$K = -\frac{\pi}{2} - 2\pi\frac{l}{T}$$

Replacing K in the first member of Equation (4 *b*) by this value,

$$A \sin\left(2\pi\frac{t}{T} + K\right) = A \sin\left(2\pi\frac{t}{T} - \frac{\pi}{2} - 2\pi\frac{l}{T}\right) = A \cos 2\pi\left(\frac{t-l}{T}\right)$$

This latter expression may be expanded into,

$$A \cos 2\pi\left(\frac{t-l}{T}\right) = A \cos 2\pi\frac{t}{T} \cdot \cos 2\pi\frac{l}{T} + A \sin 2\pi\frac{t}{T} \cdot \sin 2\pi\frac{l}{T}$$

Again substituting for $\cos 2\pi\frac{l}{T}$ and for $\sin 2\pi\frac{l}{T}$ their respective values, $\frac{h}{H}$ and $\frac{1}{H}\sqrt{H^2 - h^2}$, derived from Equation (4),

$$A \cos 2\pi\left(\frac{t-l}{T}\right) = A\left(\frac{h}{H}\cos 2\pi\frac{t}{T} + \frac{1}{H}\sqrt{H^2-h^2} \cdot \sin 2\pi\frac{t}{T}\right)$$

from which Equation (4*b*) can be written in the form,

$$A\left(\frac{h}{H}\cos 2\pi\frac{t}{T} + \frac{1}{H}\sqrt{H^2-h^2} \cdot \sin 2\pi\frac{t}{T}\right) = \frac{1}{H}(H^2 - h^2)\sin 2\pi\frac{t}{T} + \frac{h}{H}\sqrt{H^2-h^2}\cos 2\pi\frac{t}{T}$$

or,

$$A\left(\frac{h}{H}\cos 2\pi\frac{t}{T} + \frac{1}{H}\sqrt{H^2-h^2}\sin 2\pi\frac{t}{T}\right) = \sqrt{H^2-h^2}\left(\frac{h}{H}\cos 2\pi\frac{t}{T} + \frac{1}{H}\sqrt{H^2-h^2} \cdot \sin 2\pi\frac{t}{T}\right)$$

whence,

$$A = \sqrt{H^2 - h^2}$$

The equation for s can now be written,

$$s = \frac{1}{2L}\sqrt{H^2-h^2}\cos 2\pi\left(\frac{t-l}{T}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Substituting this expression for s in Equation (5),

$$v = C\sqrt{r\,s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$v = C\sqrt{\frac{r}{2L}}\sqrt[4]{H^2-h^2}\sqrt{\cos 2\pi\left(\frac{t-l}{T}\right)} \quad \ldots\ldots\ldots\ldots (7)$$

This is a general expression for the value of v at any time, t. It will evidently be a maximum when s is a maximum, that is, when $t = l$. This maximum velocity occurring at the strength of the tide is the only value of v of importance, as that velocity determines the effective scouring capacity of the currents. Representing this maximum mean velocity by V_m, and remembering that $r = \frac{a}{p}$,

$$V_m = C\sqrt{\frac{a}{2\,p\,L}} \cdot \sqrt[4]{H^2 - h^2} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

After an inlet has assumed a comparatively stable regimen, this will be its critical velocity previously described.

Substituting the value for v (Equation (7)) in the equation, $d\,q = a\,v\,d\,t$, gives an expression for the differential discharge through an inlet, in which t is the only variable.

$$d\,q = C\,a\sqrt{\frac{a}{2\,p\,L}} \cdot \sqrt[4]{H^2 - h^2} \cdot \sqrt{\cos 2\,\pi\left(\frac{t-l}{T}\right)} \cdot d\,t \ldots\ldots (9)$$

To simplify the variable quantities for integration let,

$$x = 2\,\pi\left(\frac{t-l}{T}\right) + \frac{\pi}{2}$$

then,

$$\cos 2\,\pi\left(\frac{t-l}{T}\right) = \sin x$$

and,

$$d\,x = \frac{2\,\pi\,d\,t}{T}$$

or,

$$d\,t = \frac{T}{2\,\pi}\,d\,x$$

The value of t that will make $x = 0$ when $q = 0$ is $l \pm \frac{T}{4}$. To find Q the differential must therefore be integrated between the limits $l - \frac{T}{4}$ and $l + \frac{T}{4}$; but,

$$\int_{l-\frac{T}{4}}^{l+\frac{T}{4}} \sqrt{\cos 2\,\pi\left(\frac{t-l}{T}\right)} \cdot d\,t = \frac{T}{2\,\pi}\int_0^{\pi} \sqrt{\sin x} \cdot d\,x$$

Hence,

$$Q = C\,a\sqrt{\frac{a}{2\,p\,L}} \cdot \sqrt[4]{H^2 - h^2} \cdot \frac{T}{2\,\pi}\int_0^{\pi} \sqrt{\sin x} \cdot d\,x \ldots\ldots (10)$$

By means of the so-called gamma functions, the value of $\int_0^{\pi} \sqrt{\sin x} \cdot d\,x$ is found to be 2.3962 +, hence the equation for Q may be reduced to,

$$Q = C\,a\sqrt{\frac{a}{2\,p\,L}} \cdot \sqrt[4]{H^2 - h^2} \cdot \frac{T}{2\,\pi}\,(2.3962)$$

or,

$$Q = 12\,054\,C\,a\sqrt{\frac{a}{p\,L}}\cdot\sqrt[4]{H^2 - h^2} \quad \ldots\ldots(11)$$

The area of the basin is assumed to be constant for all stages of the tide, and since its cubical contents between high and low water must be equal to the volume of water flowing through the inlet, it is clear that such volume may be expressed by the equation,

$$Q = m\,h \quad \ldots\ldots(12)$$

Equating the two expressions for Q,

$$m = 12\,054\,C\,a\sqrt{\frac{a}{p\,L}}\cdot\sqrt[4]{H^2 - h^2} \quad \ldots\ldots(13)$$

That is,

$$H^2 - h^2 = \left(\frac{m^4\,p^2\,L^2}{(12\,054)^4\,C^4\,a^6}\right)h^4$$

Let,

$$\frac{m^4\,p^2\,L^2}{(12\,054)^4\,C^4\,a^6} = \frac{1}{2\,B} \quad \ldots\ldots(14)$$

then,

$$h^4 + 2\,B\,h^2 = 2\,B\,H^2$$

or,

$$h = \sqrt{\sqrt{B\,(2\,H^2 + B)} - B} \quad \ldots\ldots(15)$$

Equations (11), (13), and (15) will give (1) the volume of tidal discharge that is capable of passing through a given inlet under given conditions in the bay; (2) the size of bay required to allow the tide in it to rise to a given height when flowing through an inlet of given size; and (3) the height to which the tide will rise in the bay.

From Equation (4),

$$\sqrt[4]{H^2 - h^2} = \sqrt{H \sin 2\,\pi\,\frac{l}{T}}$$

This second value may be substituted in Equations (11), (13), and (15) when desired.

These formulas are based on the assumption that the flow through the inlet is solely hydraulic and on other necessary assumptions fully explained. They are of general application for cases where hydraulic flow greatly exceeds any flow due to propagation of a tidal wave through the inlet, as no note of such has been taken in deriving them.

Test of Formulas.—Most of the terms of the formulas are capable of direct observation or of computation from observed data, and the correctness of all these formulas has been thus checked in the case of Absecon Inlet, New Jersey, the computed values of certain factors being satisfactorily in accord with the observed values of those factors.

The discharge at Absecon Inlet was gauged October 16, 1923, the heights of the tide being observed both in the bay and in the ocean. Observations

were made in both Main Channel (Fig. 5), which then had not been closed, and in Absecon Channel, or Inside Thorofare as it is sometimes called. Table 1 gives the observed and the computed data relating to this inlet.

TABLE 1.—DATA AND HYDRAULIC COMPUTATIONS FOR ABSECON INLET.

Feature.	Observed value.	Computed value.
ABSECON CHANNEL.		
Area, a	17 750 sq. ft.	.....
Wetted perimeter, p	910 ft.	.....
Hydraulic coefficient, C	116	.....
Length of channel, L	35 000 ft.	.....
Lag, l	1.57 hours.	1.46 hours.
Tidal prism, Q_1	944 640 000 cu. ft.	988 200 000 cu. ft.
Height of ocean tide, H	3.8 ft.	.....
Height of bay tide, h	2.8 ft.	.....
Area of bay (part), M_1	.....	335 000 000 sq. ft.
Maximum velocity, V_m	3.00 ft. per sec.	3.1 ft. per sec.
MAIN CHANNEL.		
Area, a	12 000 sq. ft.	.....
Length, L	10 000 ft.	.....
Wetted perimeter, p	1 810 ft.	.....
Hydraulic coefficient, C	85	.....
Tidal prism, Q_2	518 990 000 cu. ft.	507 410 000 cu. ft.
Area of bay (part), M_2	.....	181 210 000 sq. ft.
Maximum velocity, V_m	2.46 ft. per sec.	2.48 ft. per sec.
COMBINED CHANNELS.		
Tidal prism, $(Q_1 + Q_2)$	1 468 630 000 cu. ft.	1 445 610 000 cu. ft.
Total area of bay, $(M_1 + M_2)$	523 860 000 sq. ft.	516 210 000 sq. ft.

The computed values agree so closely with observed values as to justify the equations.

Limitations of Formulas.—All the foregoing formulas must be used with due regard to their limitations. They were deduced to cover only the case of one inlet connecting a tidal sea with a basin of definite capacity within which the rise and fall of the water is due entirely to the flow in and out through the inlet. They would not be true in case of an inlet or strait connecting two tidal seas, as, for instance, a sea-level Panama Canal, the Cape Cod Canal, or the Chesapeake and Delaware Canal; neither would they be true in case the interior basin were free of all tidal oscillation, except through the inlet, but also received a large discharge from rivers. In that case the inflow and outflow through the inlet would no longer be equal.

The value of C is constant only under the condition of a uniform channel between the sea and bay which has been assumed. In practice, the channel is seldom uniform, and the flood-tide deposits continually tend to destroy any uniformity that it may possess by decreasing the channel depths and causing the current energy to be dissipated on shallow flats. This increase in friction losses results in a diminished value of C at such places. Hence a value of

C computed for the ideal conditions within the gorge may be too large by a considerable percentage for the inner channels through and over the inner delta.

It follows that the value of C computed by either Kutter's or Bazin's formula for the gorge channel, and that computed by substitution of observed data for the other terms of the formula herein given will be different; and the latter value can only be made equal to the former by the application of a factor, k, so that, $k = \frac{C}{C_1}$.

As C is the value computed for the gorge, and C_1 that determined from the observations, k will be a measure of the increased rate of frictional resistance which the channel has acquired by reason of its failure to be uniform in depth and capacity with the gorge channel; and, therefore, may be considered as a measure of the degree of efficiency of the channel in carrying the volume of water it might be capable of carrying under the given conditions of area of basin, range of tide, size and length of channel, etc.

Another phenomenon to which attention must be called is that all the so-called constants in the equations are really variable and have a somewhat greater value at high water than at low water. The result is that at high water, particularly when h is small and maximum velocities occur at or near high and low waters, the high-water velocities will exceed the low-water velocities, but since that condition will affect both flood and ebb tides alike the inequalities are neutralized, and their consequences are negligible.

In selecting a value for h, thus determining the size of channel that an inlet should have to fill properly a given basin, consideration must be given to the fact that this value controls the difference in tidal phase of the ocean and of the bay, or the lag of the basin tide. By the usual mathematical processes, it may be shown that the slope and, consequently, the velocity is always at a maximum when $t = l$, or at a time after mean sea level on a rising tide equal to the lag of the basin tide. Should the lag reach its maximum value, $\frac{1}{4}\ T$, and, consequently, h become O, the maximum velocities will occur at high and at low water. Should h be made larger by providing a larger channel between inlet and basin, and, consequently, the lag be made smaller, the time of maximum velocities will more nearly approach the time of mean sea level. An increased value of h would allow a deeper layer of water having a greater hydraulic radius to flow out of the basin early in the ebb tide, and might tend to give it a considerable predominance over the flood tide in removing deposits. Observations seem to confirm a view that those inlets having a channel large enough to give a considerable value to h are generally in better shape than those where the tidal variation is not felt in the bay, that is, when h is at or near zero, although the latter gives the greater values for the velocity.

Friction and Other Resistances

Effect on Velocity.—Every obstruction in the inlet, whether it be artificial or natural, resulting in congestion to such a high degree that the tidal cur-

rents are not adequate to remove the accumulations of drift, operates to a certain extent as a dam in back, or landward, of which the tidal fluctuation will tend to limits that become narrower as the relative effect of the dam or the barrier becomes more and more pronounced. Thus, an inlet may form the connection between the ocean with a range of 4 ft. and a lagoon, the main body of which is not more than a mile from the ocean with a tidal range of 0.5 ft., which is the condition at Manasquan River and Inlet, New Jersey.

Friction is proportional to the square of the velocity and is in inverse ratio to the hydraulic radius, which for large sections does not differ appreciably from the mean depth. Therefore, in channels where the mean depth is considerable and the velocities are moderate, friction may have a negligible influence in comparison with the enormous energy of the current; but under changed conditions, when the depth is small and current velocities are large, friction increases greatly in relative importance.

Thus far, observations on velocities do not determine definitely the influence of energy and of friction on their variations. It is probable, however, that the energy of the current and the decrease of bottom depth have the general effect of increasing velocities, but that this result is counterbalanced by the resistance of friction, and all the more so as the mean depth becomes smaller.

Assuming a uniform slope of 1 ft. per mile and a constant width of channel, the velecity of flow will decrease with a decrease of depth as follows:

Depth, in feet.	Velocity, in feet per second.	Depth, in feet.	Velocity, in feet per second.
7.00	3.10	2.00	1.15
6.00	2.75	1.50	0.95
5.00	2.40	1.00	0.65
4.00	2.00	0.50	0.36
3.00	1.60	0.25	0.19

Effect on Tidal Energy.—The energy of the water flowing into the basin is partly potential and partly kinetic. Let W be the weight of a unit volume of water, Q, the volume of the tidal prism, and Z, the height of the center of gravity of the water above mean low-water level. The potential energy, therefore, will be, $W\,Q\,Z$. The kinetic energy at any moment will be, $\frac{1}{2}\,\frac{W}{g}\,V^2$. The total energy of the tide, therefore, will be,

$$E = W\,Q\,Z + \frac{1}{2}\,\frac{W}{g}\,V^2.$$

The energy of the ocean tide causes the heaping up of the water along the shore, and the water that flows into the inlet takes its energy from that. This total energy is being constantly reduced by friction. It can be shown that if the channel be uniform the energy will be reduced according to an exponential law of the form, $E = E_0\,\varepsilon^{-mx}$, E_0 being the original energy at the ocean bar, and m a coefficient.

At high-water slack all the kinetic energy will have been transformed into potential energy, and at low-water slack the entire energy of the flow will have been spent. It does not follow that it will have been consumed in the inlet and basin, which will consume only so much thereof as may be necessary to overcome the resistances therein, but that any remaining energy in the tide is dissipated in the ocean on its return there. It is the object of the engineer to utilize as much of this lost energy as possible to assist in the desired improvement.

Evidently, the amount of energy in the moving water is dependent on its volume, velocity, and tidal range. All are dependent on the mutual character, size, and other related physical characteristics of the inlet and basin. The energy increases more rapidly with the velocity than with the volume, but so also does the effect of friction, which causes a more rapid consumption of energy without doing useful work. Friction has a relatively smaller effect on large masses moving at low velocities than on smaller masses moving at high velocities and having equal original energies. This, together with the fact that the ratio of silt or drift to water is so much smaller with a large volume, explains why large inlets are so much more readily maintained in good condition than small ones.

Effect of Variation of Channel Width.—Since the discharge through the inlet and thoroughfare, if there be one, must pass through each cross-sectional area, the product of that area and the velocity prevailing there, say, at maximum slope, will be constant. Then $a\ v = a_1,\ v_1 = a_2,\ v_2$, etc., for any number of points. A value of a less than the average therefore requires an increased velocity which can be obtained only at the expense of the total energy, resulting in a local rise in elevation until the increased slope is great enough to increase the velocity sufficiently to produce the required discharge. The effective value of H is, of course, reduced by an amount equal to the sum of all such local rises. A value of a above the average means a decreased velocity, with consequent deposit of silt.

Propagation of Tide in the Lagoon

The projection of the discharge into the lagoon basin at the point of ingress results in a displacement of the water therein to make room for the new volume, thus giving rise to an increase in level of the basin at that point, the water already in the basin being pushed aside for its entire depth. This gives rise to a wave of translation in the basin which is propagated therein under the laws pertaining to waves of translation, which are well known. The formula of Scott Russell for such waves is, $V = \sqrt{g\ (d + h)}$, in which, V is the rate of propagation; d, the low-water depth; and h, the height of the wave. The velocity of the current produced by the wave of translation is,

$$u = V \frac{h}{d + h} = \frac{h \sqrt{g}}{\sqrt{d + h}}$$

These formulas show that the shape of, and the depths in, the basin will have some effect on its ability to receive the full quantity of water that would be indicated by the expression, mh.

Assume the case of Barnegat Bay on the coast of New Jersey. The average depth of this bay is 6 ft., and the tidal wave is 1 ft. on the inner bar, hence

$$V = \sqrt{g\,(6+1)} = 15 \text{ ft. per sec.},$$

and

$$u = \frac{h\sqrt{g}}{\sqrt{d+h}} = \frac{1 \times 5.67}{\sqrt{6+1}} = 2.15 \text{ ft. per sec.}$$

Since the time from low water to high water, or $\frac{1}{2}\ T$, is 22 350 sec., the effect of the inflowing tide would be propagated the 40 miles to the upper end of the basin in about 4 hours, while the sea water would only flow in 9 miles before it would reverse.

Since the wave of translation as it enters the bay is reversed before being transmitted throughout the more remote parts, it follows that the rise of those parts of the basin due to the inflow of the tide is not equal to h, but may be much smaller if the depth of water in the bay is small. It is, therefore, necessary to apply a correction factor for each bay to the expression, mh, in determining its reservoir capacity, making the true term, $k_2 mh$.

This correction factor will be near unity for bays of circular form which the thoroughfares enter through a delta, as in Reeds and Absecon Bays (Fig. 5) on the New Jersey coast, and will be smaller for bays like Barnegat and Chincoteague Bays (Fig. 1) which are elongated and connect with the inlet at one end. The factor will also be greater or less as the depth of the bay is greater or less. It would be large for Cold Spring Harbor, but very small for Grassy Sound on the New Jersey coast.

It is not probable that the value of this factor can be determined directly except by cubature of the tidal prism, but it can be combined with the factor, k, to be applied to the value of C as referred to previously, so as to give the final factor, k_e.

Since k has been shown to be the measure of efficiency of the inlet and thoroughfare in conveying the water over the inner bar into the basin, and k_2 is the measure of the efficiency of the basin in receiving that water, $k_e = \frac{k}{k^2}$ may be termed the coefficient of the efficiency of the combination of inlet and basin. Should both inlet and basin have an equal factor of efficiency or inefficiency, making k_e near, or equal to, unity, it would mean that no improvement of one without an equal improvement of the other would be justifiable, or that both should be improved equally should the basin be capable of enlargement.

Transporting Power of Currents

Conclusions from Experiments.—Little is known experimentally of the transporting power of large currents in open channels, an understanding of which is so necessary to an appreciation of the ability of currents to perform the work of improving channels.

An English writer,* Deacon, observing the motion of sand in a trough with glass sides, found three clearly distinct phases of sand motion:

* *Minutes of Proceedings*, Inst. C. E., Vol. CXVIII, p. 93.

First.—A discontinuous rolling motion, with surface velocities from 1.3 to 2.1 ft. per sec.

Second.—A discontinuous suspension in the lower layers of the current between 2.1 and 2.9 ft. per sec.

Third.—A continuous suspension at surface velocities of more than 2.9 ft. per sec.

A French writer,* Lechalas, considering V_0 to represent the velocity, in feet per second, at or near the bottom, V_s, the surface velocity, and V, the mean velocity, in case of sand with grains of a mean diameter of 0.04 in., found as follows: Until V_0 exceeds 0.82 ft. per sec., or V_s exceeds 1.27 ft. per sec., there is no motion. Once motion begins, V_0 cannot be directly observed, but the state corresponding to Deacon's third phase is reached with V_s equal to 3.3 to 3.4 ft. per sec.

Lechalas assumes that the force available to move the sand when V_0 exceeds 0.82 ft. per sec., is,

$$P = A\ (V_0^2 - 0.82^2) = A\ (V_0^2 - 0.67)$$

and this he puts equal to $b\ q$, in which, q is the volume of sand transported per foot width of channel, which is assumed to have a width of b ft. Thus,

$$q = \frac{A}{b}\ (V_0^2 - 0.67)$$

as long as V_0 does not exceed 3.33; that is, it is true for Deacon's first and second phases.

Lechalas found the probable value of $\frac{A}{b}$ to be 0.0004, or

$$q = 0.0004\ (V_0^2 - 0.67)$$

Scouring Velocities.—It follows that as long as the bottom velocity does not exceed what may be termed the first scouring value, which may be termed V_1, the particles remain undisturbed. When this value is exceeded, motion begins according to Deacon's first phase and gradually passes over to his second phase, but there is no abrupt change in character until the third phase is reached with a bottom velocity which may be termed the second scouring velocity, or V_2. When the bottom velocity exceeds this value the whole of the bottom layers of water are charged and clouded with sand, and this cloud rises higher as the velocity is increased.

It is plain that once the bottom velocity exceeds V_1 scour will begin in a channel, and continue until by increasing the size of the channel the bottom velocity is reduced to or below the value of V_1 corresponding to the material of which the channel bed and sides are formed.

From the law of Lechalas the absolute quantity of silt to be transported for any bottom velocity can be found and will be the same per foot of bed no matter what the depth may be as long as the bottom velocity is unchanged.

Since Lechalas' law is generally conceded to fail in the third phase of transport another expression for the relation between velocity and silt must be found for it. An examination of all available experimental data has led to the expression, $Q = 0.0000166\ V_0^5$, in cubic feet per second.

* *Annales des Ponts et Chaussées*, 1871, Vol. 1.

Based on a general formula for transport of silt in this phase of the general form, $V = C\sqrt[n]{d}$, the following general rules relating to scour are given.*

Fine silt:

Clear water scours when $v > 0.40\ \sqrt{d}$.
Silted water does not deposit when $V > 1.05\ \sqrt{d}$.
Silted water scours when $v > 1.37\ \sqrt{d}$.

Coarse sand, 0.04 in. in diameter:

Clear water scours when $v\ 16 > (d)^{0.33}$.
Silted water does not deposit when $v > 2.2\ (d)^{0.33}$.

Coarse gravel, pea size:

Clear water scours when $v > 2.0\ (d)^{0.25}$.
Silted water does not deposit when $v > 2.5\ (d)^{0.25}$.

Boulders:

Moved when v exceeds $5\ (d)^{0.25}$.

Ratio of Silt to Water.—This indicates that the deeper the channel the less likely it is to scour, and the coarser the material the less marked is the effect of depth.

Assuming a width of 1 ft. in any cross-section, $v = C\sqrt{d\,s}$ and $v = e\,d^n$, in which, C and n depend on the volume of silt carried and its mean size.

$$v = C\sqrt{d\,s} = e\,d^n$$

hence,

$$s = \frac{e^2\,(d)^{2n-1}}{C^2}$$

which may be regarded as fixing the slope at which a channel of given depth and roughness will remain in quasi-equilibrium.

If a stream carries but little silt, n will vary between 1.5 and 2.5 and the transport will be that of the first phase; a value greater than 2.5 indicates the second phase. In the third phase it may be as high as 6.

If w be the ratio, $\frac{\text{silt}}{\text{water}}$, it is found that if v is kept constant, d must increase or decrease in the same ratio as $\frac{1}{w}$ increases or decreases. This means that an excess of silt or drift for a given velocity is certain to deposit and cause a decrease of depth, while an increase in volume of water flowing with respect to the drift will as certainly promote an increase in depth, or scour. It is probable that a special study would have to be made along the foregoing or similar lines to evaluate the exact relationship between scour and velocity for any particular locality.

Scour as Affected by Character of Deposits.—At most inlets on sandy coasts the material comprising the bed of the ocean is a light fine sand admixed with a certain amount of soft mud. This mud is not likely to be found where wave action is effective. However, inside the inlets, in the comparatively sheltered waters, various admixtures of sand and mud may be expected. When

* "The Control of Water," Parker, Lond., 1913, pp. 487–495.

this mixture has been undisturbed for a considerable period, the colloidal effects of mud will begin to consolidate the mixture rendering it more difficult for the current to erode. Frequently, therefore, in inside waters a current velocity which will erode and transport sediment freshly deposited is not able to do so with sediment which has become somewhat consolidated. Thus, erosion may be expected on the ocean bar at velocities which are not necessarily erosive inside the inlet; in other words, transporting velocities are not necessarily erosive velocities, but, when erosion has once occurred or material has been put in suspension from any cause, transportation may be expected to occur at the velocities given.

Assuming an inlet of 15 ft. depth at low water, at maximum tidal flow the bottom velocities will differ from the mean by about 0.5 ft. per sec. If the first effects of erosion be considered to begin at a bottom velocity of 1.5 ft. per sec., this will correspond to a mean velocity of 2 ft. per sec., which therefore may be considered as an erosive velocity. When an inlet has reached a state of equilibrium where changes are slight and of infrequent occurrence, erosion and fill from the flood tide are equal, that is, the one is just sufficient to remove the other. If the amount of material brought in by the flood tide is small, the work of erosion to be done is small, and the inlet will have adjusted itself to a channel which will allow correspondingly gentle velocities. The inlet, therefore, will be in good condition with due regard to its possible tidal prism.

If the littoral drift is large, and correspondingly large amounts are brought in by the flood tide, the current must have a greater velocity to remove the greater quantity of drift. It can acquire that greater velocity naturally only as the cross-sectional area becomes smaller by the deposit of some of the drift, thus causing a deterioration in the condition of the inlet. It is apparent from these considerations that the critical velocity at an inlet on a sandy coast will depend on the degree of fineness of the material comprising the ocean bar and on the readiness with which it is eroded.

Hooker's Conclusions.—Mr. E. H. Hooker, after a rather complete study of available data on the suspension of solids in rivers, expresses his conclusions in the form of a series of propositions, as follows:*

"1.—The movement of solids by water currents may take place by dragging, by intermittent suspension, or by continuous suspension.

"2.—Motion in each of the three ways is increased with an increase of depth; yet the depth itself can only affect the intermittent motion.

"3.—Motion by each of the three ways is increased by increase in mean velocity.

"4.—The presence of sediment in a stream decreases its mean velocity.

"5.—Dragging, as well as suspending power, increases with the heaviness of the liquid and with its coefficient of viscosity.

"6.—By far the greater amount of solid material transported by sedimentary rivers is carried in continuous suspension.

"7.—Sand waves of considerable size are formed in the larger rivers. Their motion down stream is slow, increasing in rate up to a certain critical value of bottom velocity and then decreasing with further increase.

* *Transactions*, Am. Soc. C. E., Vol. XXXVI (December, 1896), pp. 313–314.

"8.—The fineness of material in suspension increases from the rise to the embouchure of rivers.

"9.—The weight of suspended material per cubic unit of water decreases from rise to embouchure.

"10.—An increase in vortex motion increases the power to transport.

"11.—Continuous vertical or lateral currents which could not be explained by local causes in rivers have not yet been proved.

"12.—The phenomenon of suspension requires for its explanation a continuous upward force. Intermittent forces are not sufficient, although the intensity of the force may vary.

"13.—Bodies suspended in flowing water, either intermittently or continuously, tend to acquire a velocity greater than that of the water surrounding them.

"14.—The transportation of material consumes part of the energy of any silt-bearing stream. A greater load of fine particles than of coarse can be carried with the same expenditure of energy."

Hooker's conclusions* as to the causes of the suspension of sediment in flowing water are three:

"First.—The resultant upward thrust due to eddies, conditioned on the facts that the earth profile offers more rugosities than the air profile and that the effort exerted by a current on a solid varies as the square of the relative velocity.

"Second.—The resultant upward motion of solids due to the fact that an immersed body tends to move faster than the mean velocity of the displaced water and in such motion tends to follow the line of least resistance.

"Third.—The viscosity of the water."

Preservation of Tidal Flow

Necessity for Large Volume of Flow.—Consider an inlet which has reached an apparent state of semi-equilibrium for a tidal flow of a given amount. For any less tidal flow resulting from a very low series of neap tides there might be in the shoaler reaches of the thoroughfares a portion or portions where the flood tide deposits may gain slightly over the ebb scour, which deposits will result in a somewhat greater obstruction to the next normal tide, and in a small decrease in flow. Such an upsetting of an equilibrium can only result in progressive deterioration, since at each following period of deficient tides the same action will be repeated and continued until closure results.

The maintenance of a full-sized channel by dredging between the bay and the ocean is essential to maintain the full efficacy of the currents and to cause the expenditure of their energy at the desired place. A great advantage of this interior dredging, aside from the question of maintaining good hydraulic conditions, is that it can be done so much more cheaply than on the ocean bar (except those bars on which large sea-going dredges can be used), and that the matter of disposal of the dredged material can often be combined with the reclamation of the marshes, to the great advantage of both classes of operations. Also, the character of equipment required for the interior dredging is simpler and less expensive to acquire than for ocean-bar dredging in small inlets.

* *Transactions*, Am. Soc. C. E., Vol. XXXVI (December, 1896), pp. 322-323.

Considering that the amount of littoral drift brought within reach of the flood tide of an inlet is independent of the size of the inlet, and will be the same in volume or will vary in volume from causes outside the inlet, whether the inlet be large or small, it must be apparent that the larger the volume of current flowing, the smaller the ratio of drift to volume of flow, and hence the more easily is deposit taken care of and the formation of shoals and sand banks detrimental to navigation and to tidal flow prevented.

At the Third International Congress of Navigation held at Frankfort-on-Main, Germany, in 1888, Franzius, in discussing the importance of increasing the volume of tidal flow, expressed the view that:

"Estuaries without marked tidal fluctuation can in most cases only be rendered navigable by narrowing-in and keeping together the fixed and non-increasable amount of flow in all reaches where the formation of branches or deltas is to be feared, and by then carrying it out to deep water between training banks or jetties. Should a natural lagoon, separated from the sea, and filling and emptying under the influence of varying winds, exist close above the lowest reach, or at the point where the river proper flows into the estuary, this can be utilized for obtaining a greater depth for the lower reach (the estuary proper) than is attainable above. Estuaries having high tides may be rendered navigable by increasing the quantity and velocity of the tidal column as far as possible * * *. The average velocity, however, in the main stream or in the low-water channel must not be permitted to diminish for fear of the formation of a bar. The low-water channel should be uniform, carried along on bold lines, and as far as possible bounded on each side by a low training wall."

At its Fifth Session, in Paris, in 1892, that Congress in considering the effects of tidal flow in estuaries, expressed conclusions applicable also in the present case, as follows:

"The size and depth of a tidal river being mainly due to the tidal flow, any works which increase its volume and extend its influence, such as the removal of obstructions, dredging hard shoals, and the lowering of the low-water level by deepening the channel, effect an improvement in the navigability of the river; whilst any works which restrict the tidal influx, even though producing a local deepening by scour, are liable, unless under exceptional conditions, to injure the general navigable capabilities of a tidal river.

"The regulation of the banks of tidal rivers, so as to remove abrupt variations in width, equalises the tidal flow, reduces accretion, and facilitates the tidal influx, and therefore constitutes an important means of improvement, even if accompanied by a slight reduction of tidal capacity at certain parts by the obliteration of indents, which are in general more than compensated for by the improved scour, and consequent lowering of the low-water line, especially if accompanied by the removal of shoals.

"The extent of the filling basin necessary for the maintenance of rivers and of their estuaries depends more on the methodical and rational laying out of the sections and widths, than on the lateral reservoirs which sometimes present grave inconveniences, and which ought only to be constructed in special cases."

The importance of dredging in the inner delta channels as a means of conserving the tidal flow of the inlet and preserving it against self-destruction is thus manifest.

The Inner Bar the More Obstructive to Tidal Flow.—The principle of concentrating the currents into narrow and deep channels as far as practicable

is usually found to be grossly violated under natural conditions both at the inner and the outer deltas or bars of inlets. The outer bar violates the rule with its great cross-sectional area, its long wetted perimeter, and its shallow depth. Yet, the outer bar is more easily affected by the forces of the ocean which may remove much of the deposits there, preventing its undue extension into the ocean and making it quite short across from deep water inside to deep water outside. The inner bar usually retains more deposits than the outer and is affected destructively only by the ebb tide. It therefore builds up faster, is more extensive, the distance across from deep water in the gorge to deep water in the bay is greater, and it is more likely to be intersected by numerous small channels separated by intervening flats.

The growth of the interior delta may become so great that long channels, sometimes called thoroughfares, which are really continuations of the inlet gorge, are found connecting the bay with the inlet, the actual inner bar (or point of least depth) being where these thoroughfare channels enter the bay. Both bars are obstructive to the inlet currents, and, in view of the preponderence in extent of the inner bar, it is logical to assume that it is a greater obstruction to those currents than the outer bar. Yet in inlet improvement projects scant consideration is usually accorded to the inner bar and to the increase of inlet current velocity and discharge possible as a result of its improvement.

The rapid dissipation of current energy and velocity in intricate channels and flats, even when they are just inside the inlet, is clearly shown in the case of numerous inlets of small size and importance connecting the ocean with large bays, which, considering the total volume of water the bays are capable of receiving and discharging under the tidal range prevailing, should maintain much better conditions on the outer bars of those inlets than now exist. Barnegat Inlet on the New Jersey Coast is an example, and Ocracoke Inlet on the coast of North Carolina is another.

Barnegat Inlet is the only outlet to the ocean from Barnegat Bay, the largest bay on the New Jersey coast, yet several other inlets maintain better velocities and discharge and, consequently, have better channels across the bars. Ocracoke Inlet is the principal outlet to the ocean of Pamlico and Albemarle Sounds, with several large rivers discharging into them, yet the numerous channels and shallow flats of both the inner and outer deltas make it worthless as a navigable waterway.

The currents into and out of either of these inlets are not sufficient to cause much perceptible variation in the level of their respective bays. In both cases, if a suitable single channel were provided through the inner bar and the currents thus confined, and not allowed to become dissipated, extending from the gorge to deep water in the bay, the greater discharge and velocity that would result would soon erode a good navigable channel on the outer bar. A further increase in the size of these inlet channels sufficient to admit enough water to cause a tidal variation of 2 ft. or more in the bays would result in an enormous volume of tidal flow, with irresistible scouring energy.

It would seem logical to assume that, in view of the principles herein brought out, and under favorable conditions, the improvement of a small inlet

for navigation by dredging and training properly applied on the inner bar, so as to make the volume of tidal discharge the maximum that could be economically secured, would yield results as beneficial as, or possibly more beneficial than, could be obtained by an equal amount of work applied on the outer bar. The objective would be to prevent the energy of flowing water from dissipating itself on the inner bar and permit it to be applied to the outer bar and thus reduce the necessary dredging operations there, which are difficult, expensive, and unsatisfactory.

If the requirements of navigation call for a channel of greater dimensions than can be maintained by the volume of water flowing, controlled by concentration on the inner bar, the further enlargement of the channel by dredging would be of doubtful economical feasibility, since after the bay tide is increased to a certain point by such dredging, that is, when h has a value of approximately $\frac{3}{4} H$, any further dredging may lessen the velocity. If, however, a method be found to give a proper increase to the volume of water flowing, the larger dredged channel may become desirable.

An examination of the formulas shows that an increased discharge must be obtained, either by an increase in the capacity of the inlet channel so as to increase h, or by an increase of the area of the bay, M, or by both means, care being taken not to make the value of h too near the value of H, as that would result in a decrease of velocity. An increase of area, M, is preferable, keeping h at the correct value to give the critical velocity. Often it will be impracticable to increase the bay area at all, but sometimes adjacent inlets or channels connecting them with their bays may be closed and their basins connected with the chosen inlet by suitable channels, the size of which can easily be computed in each case to permit the most effective use of the basin area. In some cases the discharge of an inlet might be thus increased 50%, or more.

Canals

The principles herein set forth are also applicable to an approximate degree to that class of sea-level canals which connect a tidal sea with a large bay, lake, or non-tidal sea, the essential elements being that the water level at one end of the canal be fixed, that the other end be tidal, and that the canal be short. The flow through such canals will be largely hydraulic and will conform to the laws for hydraulic flow.

The characteristics of two such canals may be cited. The Inland Waterway from Pamlico Sound to Beaufort Inlet, North Carolina, was completed in 1909. It is a sea-level canal, and when completed was 10 ft. deep at mean low water, 90 ft. in bottom width, and with side slopes of 1 on 2. It runs practically north and south, connecting Beaufort Inlet at the southern end by way of Newport River or Bay, and Core Creek with Pamlico River *via* Adams Creek. There is a tidal variation of 3 to 3.5 ft. at the mouth of Core Creek, where the dredged cut begins. Adams Creek and Pamlico River are non-tidal; their surfaces, however, are subject to changes in level due to the winds, changes of 1 ft., or more, during the day being quite usual. Based upon mean

low-water datum in Beaufort Inlet, mean low water in Adams Creek was 1.4. Mean creek level is somewhat higher, but its exact value is not known at present.

On the completion of the dredging operations in the construction of the canal some rather crude observations were made of its hydraulic properties. Tidal oscillations were determined at approximately equal intervals along the canal and current velocities measured with current meters by the so-called "six-tenths rule", the velocity at the center of the channel at that depth being considered as the mean velocity.*

It was found that the flood-tide currents generally ran somewhat longer than the ebb currents, and possibly were somewhat stronger, resulting in the hitherto fresh water in Adams Creek becoming somewhat brackish. Under the theory of hydraulic flow the currents should have their greatest strength at high and low water; it was found that it was sometimes as much as one hour after high or low water before the current was strongest, while flood and ebb slack lagged equally behind mean sea-level stage of the tide, when they should occur according to theory. This lag of the currents behind the slope is, of course, easily explained by the inertia of the flowing water, which tends to flow in one direction until a contrary head develops, sufficient to change it. However, in a canal of such dimensions the flow is not wholly hydraulic, but is partly true tidal propagation. It was found, nevertheless, that the average maximum and mean velocities and the total discharge, or tidal prism, agree very well with those computed by these formulas when h is made equal to zero.

An excellent account of the currents in the Suez Canal, which confirms these principles, has been given† by the late Baron E. T. Quinette de Rochemont, M. Am. Soc. C. E., and M. Henry Desprez.

If accurate results are desired in case of such canals as these, however, uniform flow formulas will not suffice. Wave propagation must be taken into consideration, leading to an entirely different set of formulas which are not considered here.

Flow Over the Ocean Bar, and Its Improvement

As soon as the channel passes from the gorge toward the outer bar a great increase in area and in wetted perimeter begins, and continues until the crest of the bar is passed and the channel is merged into the sea. Since such an increase results in a decreased hydraulic radius, or mean depth, and a dissipation of the energy of the flowing current, a concentration of the current even out over the ocean bar may become necessary to provide sufficient depth for navigation purposes.

Consider a continuous profile along the highest parts of the ocean bar from shore to shore. The bar will act like a submerged dam and flow will occur over all parts of it below the water surface, at rates dependent on the difference of heads, and on the depth of water. Since the navigable channel or deepest water will extend over only a very small part of the whole length, a very small percentage, perhaps less than one-half the total tidal discharge,

* *Professional Memoirs*, U. S. Corps of Engrs., 1912, p. 216.

† "Cours de Travaux Maritimes," Vol. I, p. 73, Paris, 1900.

may flow through such bar channel or pass. When the total tidal flow is known, and the bar crest profile can be found, the percentage of flow through the limits of the pass may be computed sufficiently closely in any given case to determine the approximate degree of dissipation of current energy on the bar, and thus gauge the necessity for increasing its concentration.

Dredging alone will generally be insufficient to cause a greater percentage of the discharge to flow over a selected portion of the bar, hence bar dredging in general will not result in permanent improvement of an ocean bar, unless the dredged channel is made large and deep enough to concentrate the discharge and generate currents of a proper velocity to keep it clear of deposits.

The most desirable improvements of an inlet would probably be those for the purposes of fixing a migratory inlet in place, of preventing erosion of the shores, of promoting or increasing the navigability of the inlet, or of introducing larger volumes of salt water into the lagoons to promote the fishing, oyster, or clam industries. The purpose of the improvement will have much influence on the means to be selected to carry it out. The more usual methods are jetties or dikes to confine or fix the channel and to reduce width, and dredging to increase width or depth of channel, or such a combination of these means as circumstances may require.

Improvement by Dredging

Within certain limits improvement of the ocean bar of an inlet by dredging may be the most economical and efficient method. Its use presupposes that the inlet is subject to division or to migration to a very small degree, or not at all; that the principal obstruction to the flow of the tide is within the inner delta; or that when a suitable channel is provided, the basin will be sufficiently large to receive the volume of water essential to maintenance of the necessary channel by its velocity of inflow and outflow. While dredging on the outer bar may afford temporary relief to navigation it cannot in the nature of things afford a permanent improvement in depth unless the channel and inside basin are such as to promote an increased velocity so as to maintain the greater depth.

Any dredging done either inside or outside the gorge is useless in so far as permanence of the work is concerned unless it results in improved hydraulic conditions to a degree that will either establish a state of equilibrium between scour and deposit, at the desired stage of improvement, or result in some excess scour.

Generally dredging cannot improve the outer bar of an inlet up to the full limit of capacity set by the mutual relation between inlet and basin. This can only be done by means of jetties on the outer bar usually assisted by dredging, possibly both on the outer and inner bars.

Jetties

Kind.—Many types and varieties of jetties have been used, but that found by experience to be the most useful and desirable for general use is the rubble stone mound type, of size and construction suited to the special locality and problem. The jetty is particularly suited for use when its purpose is to fix

in place a migratory inlet, to prevent erosion of the shores of the inlet, or to promote the navigability of the inlet. The length of jetties and their distance apart depend on the depth of channel to be secured, and the amount of discharge through the inlet. All these factors are susceptible of separate determination for each locality. Some jetties have been constructed of wooden piles and stone, or entirely of closely spaced wooden piles. No timber construction, however, can be recommended on the seashore on account of its short life, as it is immediately attacked by the teredo and other sea worms and is of insufficient strength to support the ravages of storms. A single jetty for an inlet has frequently been proposed, but twin jetties will be found to be more satisfactory, except in a few cases.

Functions.—The principal functions of a jetty are two: First, to prevent as much as possible the heavily laden littoral currents which flow close inshore from entering the inlet; and, second, to concentrate the energy of flow on the outer bar.

The flow of the early flood tide is usually felt along the sides and bottom of the inlet channel even before the ebb has stopped flowing in the deeper part. This young flood current is always regarded as the heaviest carrier of silt. It may enter over and across shallow flats where wave action keeps the sand continually stirred up and in suspension, causing the introduction of excessive quantities of sand, which, when washed out by the ebb tide, is immediately re-deposited on the bar, and the operation becomes a continuous cycle. If a properly constructed jetty is located so as to cut off this route of entrance of the flood tide and to compel it to enter by the deeper main entrance, the flood will no longer be charged with this abundant load of sand, and the energy of the ebb currents may be devoted to scouring an increased channel rather than being devoted entirely to overcoming the harmful excessive flood deposits.

If only one side of an inlet is bordered by such shoals, a single jetty cutting off all flow may be successful in enabling the currents to form a channel of the desired characteristics. If it does not, then the jetties must be designed to perform the other duty, that is, so to concentrate the flow on one part of the bar as to secure the maximum depth and width of channel desired, or possible for the locality.

Considering the ocean bar as a submerged weir, if this weir be shortened, an increased difference of head on each side will generate currents of velocity sufficient to erode the weir and deepen the channel. The desired depth of channel being known, the proper width between twin jetties can be computed from the formulas given, by solving for p when a proper value of a is known.

Effect on Beach Drift.—On beaches composed of very mobile material the transportation of which is generally easy and rapid, and of which the relief is consequently modified in a prompt and perceptible manner under the action of the works which may be carried out there, the placing of jetties becomes a particularly delicate matter; it is necessary to foresee especially the influence that the projected works will have on the maintenance of depth in the entrance and the vicinity of the site.

The sand which composes the beach is constantly in movement under the action of the waves and currents. The waves are the principal agents of dis-

turbance of the sands and their action is much greater than that of the tidal currents the velocity of which is generally small. Even at those places where the profile of the beach is in equilibrium, the materials of which it is composed are continually in motion.

By constructing upon such a beach works projecting out beyond the shore line, the movement of drift is modified, but the importance of this modification varies according to the forms and to the orientation of the works.

Sand accumulates in the angle between the beach and the jetty by which its travel is stopped. The littoral current, moreover, loses velocity on approaching the work to turn along its outer face to reach the open which favors deposit. From this double cause the contours in the vicinity of the work are deflected, presenting their concave side to the sea.

The building up of the shore produces an advancement of the foreshore. For a certain time the contours of certain depths may not be affected by the advancement of the shore line. The slope of the foreshore becomes then progressively steeper until it reaches the maximum slope compatible with the equilibrium of the materials which form it; a lessening of depth is thus necessarily effected, and the sand tends to extend out to the head of the jetty and to form deposits outside of and beyond its end. The benefit previously received diminishes, and, at times, even disappears; as a consequence, the work may even have aggravated the preceding condition. The calm which the jetty produces is also favorable to the deposit of drift held in suspension by the waves.

Orientation.—The various results are dependent on the greater or less lengths of the works, their orientation, the direction of the storm waves, and the strength and the direction of the currents. When a jetty is built perpendicular to the coast and its head, located in moderate depth, happens to be where the littoral current is weak, the sand, after having filled the space above the jetty, is carried by the current formed along the exterior face of the jetty. This current in meeting the littoral current partly loses its velocity and a part of its drift is deposited, tending to form a bank beyond, or possibly slightly down stream from, the front of the entrance. Another portion of the drift is carried inside by the currents, which form eddies in the angle protected by the jetty and deposit there, shoaling it.

When the jetty, in place of being perpendicular to the beach, forms an obtuse angle with it, and its outer end is curved in such a way as to be terminated by a portion almost parallel to the shore, the meeting of the longside current and of the littoral current is accomplished under better conditions. The deposit caused by the meeting of the two currents is less; it may almost cease if the littoral current immediately at the ends of the jetties is sufficiently strong to carry the drift beyond the jetties.

When numerous and violent storms attack the beach which has formed above the jetty, their action has the effect of carrying offshore considerable quantities of material, and if the offshore depths are sufficiently great this may have a favorable effect without any harm being done to the entrance. Natural circumstances, therefore, may combine in such a way as to approxi-

mate a state of equilibrium of the shore, under the sole action of natural forces, such a state corresponding to sufficient depths in the channel for navigation, but this is a rare case and usually some dredging is required to secure the desired results.

High vs. Low Jetties.—Jetties giving access to inlets and intended to fix the channel and to guide the currents may be high or low. Low jetties are built in such fashion that their crests are only slightly above the beach and, like it, have but little slope. Such jetties generally have a rounded form, with flat slopes on the sides, in order not to develop a scour at their foot; the top of these works is placed a little above the level of low water in order to facilitate their construction and repair.

High jetties completly shut out the sea; they are built to such an elevation that the waves surmount them only occasionally in storms. Often the top is fixed at least 8 to 10 ft. above the level of high water and may be surmounted by a parapet.

Low jetties disturb very little the regime of the currents and the propagation of waves. They permit a certain amount of wave action in the channel. Low jetties on sandy beaches permit to pass a certain quantity of drift which then falls into the channel; it is necessary to remove this by dredging.

High jetties stop the sand and thus cause an advancement of the beach, but this action may be remedied by dredging a guard ditch outside the approach channel in the direction from which the drift comes. They may also cause a certain amount of deposit of mud in the inner channels by reason of the calm which they cause. These jetties must be built very solidly. They, therefore, cost more than low jetties, but they give better shelter to vessels using the channel. High jetties also may guide the swell which penetrates the channel into the harbor.

Low jetties and high jetties, therefore, present reciprocal advantages and disadvantages which would lead to a preference for one or the other system according to local circumstances. In England and the United States high jetties are almost exclusively used for navigable channels. This solution is to be preferred in most cases.

Damage to Adjacent Beaches.—Since the construction of jetties is very apt to have a retro-active effect on the adjacent sea beaches, which is a matter of importance due to vested interests and high real estate values, their probable effect must be very carefully studied. The effect of such jetties as have been most carefully watched seems to be a building up of the beach on their windward side, and a certain amount of denudation of the beach on their leeward side.

This effect is quite apparent in the case of the twin jetties constructed at Cold Spring Harbor, New Jersey (Fig. 8). The beach known as Two-Mile Beach on the easterly or windward side has built out very rapidly, as much as 2 000 ft. alongside the jetties, the advance growing less at more remote points. On the westerly or leeward side a recession of the beach of 100 to 200 ft. has occurred, near the jetties, and similarly diminishing in extent at remote points. This portion of the beach has been deprived of its normal supply of

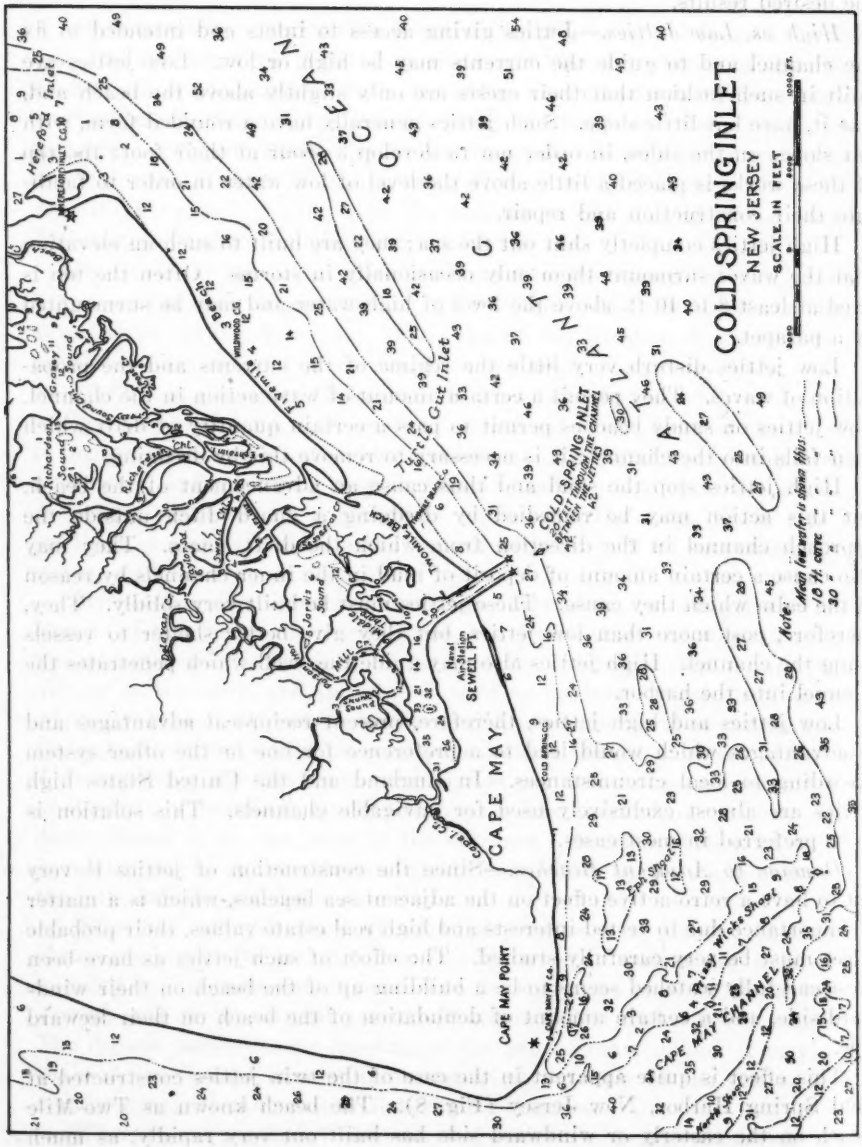
COLD SPRING INLET
NEW JERSEY
SCALE IN FEET
ATLANTIC OCEAN
Turtle Gut Inlet
COLD SPRING INLET
CAPE MAY
CAPE MAY POINT
Naval Air Station
SEWELL PT
Cold Springs
Mean low water is shown thus:
10 ft. curve
30 " "
Note:

FIG. 8.

sand, and recession has bared the underlying marsh formation at and near the low-water line. Not all the littoral sand is retained on Two-Mile Beach. The jetties deflect the currents that would carry it past the inlet into water so deep that it is no longer carried shoreward by the waves, but into deep water at sea.

Should local conditions preclude the risk of conditions likely to result from jetty construction, the problem may still be successfully solved by building dikes in prolongation of the beach line so as to narrow the gorge to the size required by the tidal flow. This would result in fixing the inlet in place and protecting it from migration or division without interruption to littoral drift, but with little or no effect on its navigability.

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PAPERS AND DISCUSSIONS

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THE COMPRESSIBILITY OF SAND-MICA MIXTURES

By Glennon Gilboy,* Jun. Am. Soc. C. E.

Synopsis

The results of a series of experiments on the compressibility of different mixtures of sand and mica, of uniform composition and grain size (0.5 mm.), form the basis of this paper. The experiments show that the presence of the flat mica grains has a marked effect on the compressibility of the mixture under load, on its expansion after release of pressure, and on the limiting voids ratios consistent with equilibrium. They indicate that the characteristic behavior of fine-grained sediments, such as silts and clays, is principally ascribable to the presence of flat grains. It is found that the compressibility characteristics of any given fine-grained soil may be reproduced on a macroscopic scale by selecting a proper mixture of coarse sand and mica. This fact indicates that any system of analysis or classification of soils which neglects the presence and effect of the flat-grained constituents will be incomplete and erroneous. Inasmuch as a direct determination of the exact quantity of flat grains in a soil is a practical impossibility, one is at once led to the conclusion that analysis and classification of soils can be of value only when based on observations of the physical characteristics of the soil as a mass, as well as the behavior of the soil under stress.

Introduction

The easiest and most obvious method of analyzing a soil sample is to pass it through a series of screens, separating it into fractions of different grain diameters. Wiegner† successfully extended this method to analyze the finer-grained fractions by sedimentation, and attempts have been made recently, with more or less success, to determine the percentage of colloidal material in the very fine-grained samples. It is a well-known fact that the individual

Note.—Written discussion on this paper will be closed in August, 1928.

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† G. Wiegner, "Ueber eine neue Methode der Schlämmanalyse," *Zentralblatt für die gesamte Landwirtschaft*, Bd. I (1920), No. 1. See, also, "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., Appendix A.

grains of any one fraction differ from one another in mass, some being bulky (approximately spherical) and others quite flat (approximately disk-shaped); but no attempt has been made to distinguish between these two classes, because there is no obvious way of separating them. Grengg and Kammel,* of Vienna, did try to determine the percentage of flat grains in a fraction by counting them under the microscope, but this procedure was tedious and inaccurate, and brought no worth while results. So that there are very complete methods for determining the grain size, but such methods entirely ignore the existence of grains which are thin in proportion to their diameter.

In 1919, Charles Terzaghi, M. Am. Soc. C. E., proposed the investigation of foundation problems by a new method.† He advocated a study of the physical properties of soils from an experimental standpoint, with a view to accumulating sufficient empirical data of all kinds to permit eventually the formulation of a complete and rational system of soil mechanics. Two years later, he published the results of his first tests on the compressibility of soils at lateral confinement.‡ These tests clearly showed the characteristics of the compression and expansion curves, and the enormous difference between the compressibility of sand and that of clay. In addition, this paper contains the first attempt at mathematical analysis of these curves, an analysis which shows that the curves for both materials are of the same form, the equations differing by certain constant factors. In 1925, Professor Terzaghi published a complete treatise on soil mechanics,§ wherein will be found an analysis of the compressibility of sands and clays and of the relationship between compressibility, bearing capacity, and settlement characteristics.

It is apparent that this progress toward a more accurate knowledge of settlement phenomena is of major importance to the engineer interested in foundations; for the most essential requirement of a well-designed foundation is that it should provide for equal and uniform settlements in all parts of the building. The next steps followed in logical sequence. In 1925, Professor Terzaghi demonstrated the impracticability of the methods suggested by Grengg and Kammel, by showing the extreme difficulty of accurate testing and the impossibility of clearly representing all factors which influence the behavior of sands.‖ In this connection he mentioned the four most important of these factors, namely, grain size, uniformity, volume of voids, and mica content. It is interesting to note that this paper contains the first hint that the mica content of soils might have an appreciable effect on their behavior.

Finally, in the spring of 1926, Professor Terzaghi suggested to the writer the advisability of making a series of preliminary studies of the effect of

* R. Grengg und K. Kammel, "Charakteristik einiger Wiener Bausande nach äusseren Kennzeichen," *Zeitschrift* des Oesterreichischen Ingenieur- und Architekten-Vereines, 1924, Heft 43-44.

† "Die Erddruckerscheinungen in örtlich beanspruchten Schüttungen und die Entstehung von 'Tragkörpern,'" Sonderdruck aus der *Oesterreichischen Wochenschrift für den öffentlichen Baudienst*, Jahrgang 1919, Heft 17-19.

‡ "Die physikalischen Grundlagen des technisch-geologischen Gutachtens," *Zeitschrift* des Oesterreichischen Ingenieur- und Architekten-Vereines, Heft 36-37, September, 1921.

§ "Erdbaumechanik auf bodenphysikalischer Grundlage," Franz Deuticke, Leipzig u. Wien, 1925.

‖ "Zur Charakteristik der Bausande," *Zeitschrift* des Oesterreichischen Ingenieur- und Architekten-Vereines, Heft 33-34, 1925.

varying mica content on the compressibility of sands.* This paper is a résumé of the results obtained and the conclusions drawn therefrom, condensation being accomplished by the omission of certain details without which the original thesis would not be complete. Even the most superficial examination of the results will convince the reader that the shape of the grains, a factor hitherto ignored, has an almost unbelievable influence on the compression characteristics of a granular material. In fact, by simply adding mica it is possible to transform a sand from a dense, almost incompressible mass into a loose mixture of flat and bulky grains which is more compressible than the fattest clay known.

It is further possible to obtain a mixture of cohesionless granular materials which will not only be as compressible as a given sample of clay, but will also reproduce the characteristic variations in compressibility and elastic expansion in all their essential details. The significance of this fact is obvious, for it shows that the compressibility of clay is not an inherent property of the material, nor is it directly ascribable to the small size of the grains, the presence of water in the voids, or the presence of particles of colloidal size. The compressibility of fine-grained sediments is undoubtedly influenced by all these factors, but in its essence it is a structural phenomenon, and as such is capable of duplication on a macroscopic sale.

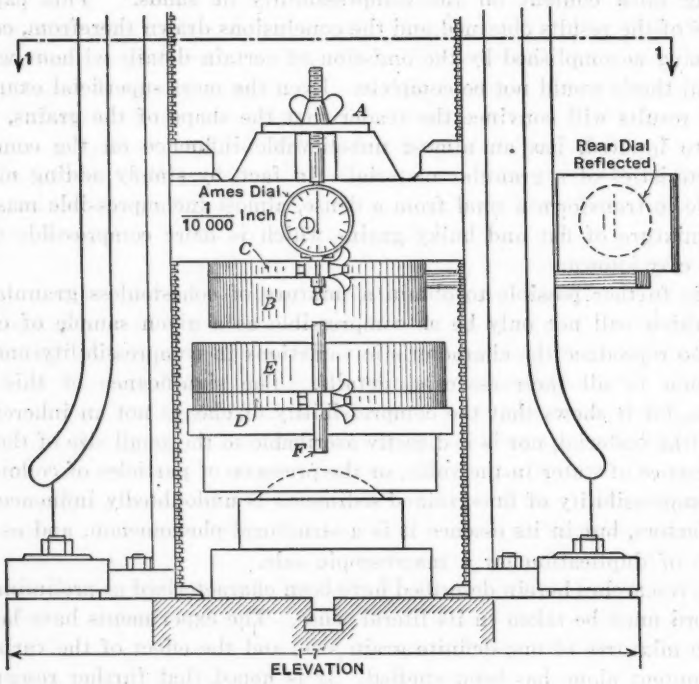
The researches herein described have been characterized as preliminary, and this word must be taken in its literal sense. The experiments have been confined to mixtures of one definite grain size, and the effect of the variation in mica content alone has been studied. It is hoped that further research will be done along this line, with other factors variable, in order to accumulate data sufficient for a thorough understanding of compression phenomena in granular materials. However, the tests already made will prove extremely useful as a guide to proper test methods and analysis of results, and will furnish a convenient basis for more elaborate investigations.

Apparatus, Material, and Procedure

Testing Machine.—The machine used throughout this work was a Riehle Universal hand-operated screw testing machine, reading by 2 lb. up to 10 000 lb.

Sample Container.—Fig. 1 shows the testing apparatus as assembled in the machine. The material to be tested is enclosed in a steel ring, *E*, provided with a base, *D*, the two parts being screwed tightly together. The base carries two integral wings, 180° apart, fashioned into split clamps to hold two short steel studs, *F*. Load is applied through a bearing block, *B*, fastened to the movable head of the testing machine by means of a long stud screwed into the block at the lower end and provided with a wing nut at the upper end. This stud passes through the hole in the head of the machine, and the wing nut rests on a steel plate, *A*, large enough to cover the hole. Between the block and the head of the machine is inserted a special gauge holder, *C*, so devised that the Ames gauges will be directly over the two steel studs projecting upward from the base, the movable spindles of the gauges resting upon the tops of the studs. In other words, the gauges move with the head of the machine, while the ends

* A complete record of these researches is contained in a thesis entitled "An Experimental Investigation of the Compressibility of Sand-Mica Mixtures," presented by the writer to the Faculty of the Massachusetts Institute of Technology, in 1927.



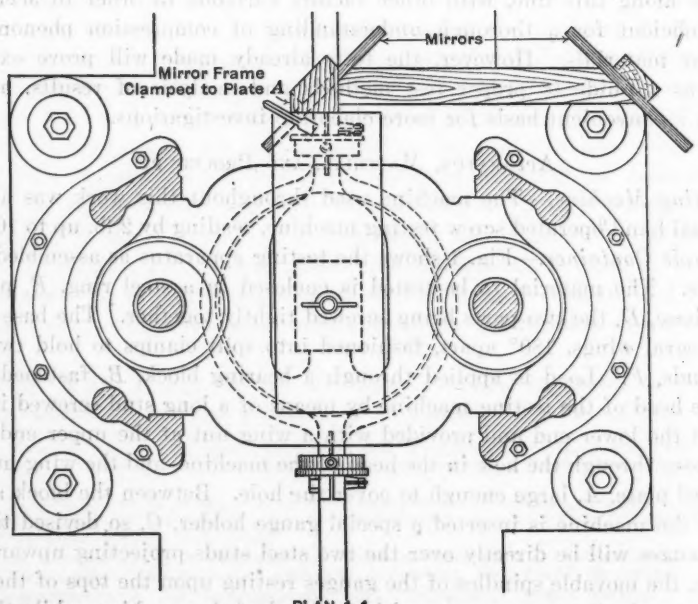


FIG. 1.—ASSEMBLY OF COMPRESSION APPARATUS.

of their spindles remain stationary, so that the average of the dial readings gives the true movement of the center of the bearing block with reference to the base of the container. A simple periscope, consisting of two plane mirrors in a wooden frame, is clamped to the projecting end of Plate *A*, and serves to enable the operator to read the dial on the far side of the machine without leaving his seat.

Sand.—These tests were conducted exclusively on a wind-worn dune sand, with smooth round quartz grains. In a mass it has a light brown, coffee color; the nearest standard color is called by Ridgway* "cream buff". This sand was chosen on account of its cleanliness, hardness, uniformity of composition, and freedom from dust and impurities. The original quantity of sand was run through a series of Tyler standard sieves, in a Ro-Tap machine, and that fraction passing a No. 28 sieve and retained on a No. 35 sieve was used in all the tests. The sample, therefore, had a grain size ranging from 0.417 to 0.589 mm., average 0.5 mm., and a uniformity coefficient of 1. The specific gravity of the sand was found to be 2.66.

Mica.—The experiments made use of a pure white mica, known commercially as Ground White Mica No. 6. The grains are angular, and quite thin; their thickness is of the order of 0.02 mm. The mica was subjected to the same screening process as the sand, and the same fraction (0.417 to 0.589 mm.) was used in all the tests. The mica was found to have a specific gravity of 2.85.

Preparation of Sample.—Mixtures were made by weight, sufficient of both materials being taken to yield enough of the mixture to more than fill the container. The mixture was loosely poured into the container and heaped up in the center in the form of a cone. The container was then thoroughly shaken, rolled over a layer of coarse sand on a table to produce a large number of small shocks, and was then hammered; the process was continued until consolidation by this means seemed to reach a limit. Then the sample was made exactly flush with the top of the container by means of a straight-edge, and the weight of the container plus the sample was determined. All weighings were made to the nearest 0.1 gramme. The container was placed on the universal block in the testing machine, the whole mechanism was accurately centered, the studs were adjusted to give small positive initial readings of both dials, and the test was ready to begin.

Testing.—The load was applied in definitely predetermined increments. These increments were found, by experimenting with the machine, to produce no appreciable time lag of compression. First, the weight on the beam of the machine was run out to the desired load, then the hand-wheel was operated until the beam lifted to a position of equilibrium. The readings of both dials were then recorded, and the next increment was added. The first compression was continued up to a load of 1 600 lb., which is a little more than 3 kg. per sq. cm. Then the load was decreased to zero; increased again to 5 000 lb., somewhat greater than 10 kg. per sq. cm.; decreased to zero; and, finally, increased once more to 4 500 lb. to give a complete hysteresis loop for the final rebound. Each test occupied 1 to 2 hours from the time the sample was placed in the machine until it was removed. The data immediately avail-

* "Color Standards and Nomenclature," the Author, Washington, D. C., 1912.

able at the end of the test were as follows: Percentage by weight of constituents of mixture; net weight of sample; varying increments of load, in pounds; and dial readings, in ten-thousandths of inches, corresponding to each increment of load.

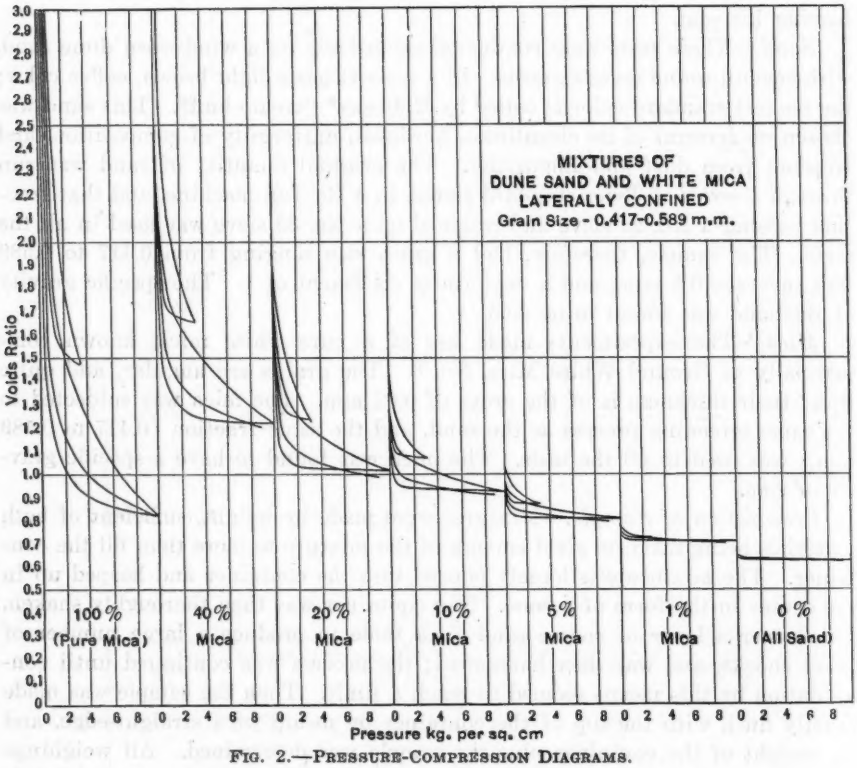


FIG. 2.—PRESSURE-COMPRESSION DIAGRAMS.

TEST RESULTS

The information derived from each test was plotted as a curve of variation in voids ratio with pressure for the particular sample tested, and these curves are included in detail in the original thesis. As the comparative values of compressibility are especially stressed in this paper, it has been deemed advisable to substitute for the detail diagrams an assembly drawing which will illustrate the important points in a more direct manner. Fig. 2 shows a set of representative curves for varying mica content, all plotted to the same scale. The following deductions may be made: First, that the pressure-compression diagram for materials at lateral confinement has a certain very definite character, irrespective of the character of the material; second, that the voids ratio of uncompressed granular masses increases with increasing percentage of flat grains; and, third, that the compressibility of these masses increases enormously with increasing mica content.

The truth of the first statement is not immediately apparent from Fig. 2. The individual diagrams, however, were originally plotted to different scales of

voids ratios according to the compressibility of the samples. All these curves are so alike in character, and, in addition, resemble so closely those of clays* that if a person were to be handed a diagram of this kind with only the scale of pressure marked thereon, he would be quite unable to decide whether it referred to a fat clay, a compact sand, or a 40% sand-mica mixture.

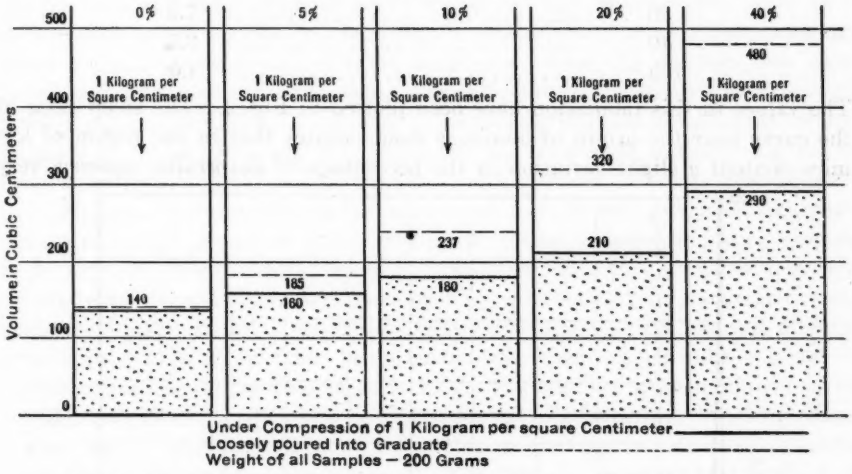


FIG. 3.—VOLUME OCCUPIED BY A STANDARD QUANTITY OF SAND-MICA MIXTURE.

The second and third statements are clearly illustrated in Fig. 2. The same effect is shown from a slightly different point of view in Fig. 3. The figures on the horizontal lines indicate the level of the upper surface of the sample as read from the graduate, in cubic centimeters, before and after the pressure was applied. The variation in initial voids ratio and in compressibility are thus brought into striking prominence. It must be borne in mind, however, that this diagram is for illustrative purposes only. The results of the simple pressure operations shown therein are not to be compared quantitatively with those of the original tests, because the height of the sample in the graduate is so great in proportion to its diameter that a considerable portion of the superimposed load is transmitted to the glass by friction; the pressure acting on the bottom of the sample is, therefore, very much less than 1 kg. per sq. cm., and the compression of the material is altered in consequence.

In attempting a mathematical analysis of the results of the tests, it was considered advisable to determine for each material a single constant which would express as closely as possible the elastic properties of the mixture. The simplest and most useful was thought to be the "coefficient of elastic expansion" ("Schwellbeiwert"), as defined by Professor Terzaghi.† In his original experiments he found that the modulus of elasticity was approximately proportional to the pressure, and that the constant of proportionality varied for different materials. This constant was determined for the sand-mica mixtures, with the following results:

* See Fig. 2, and "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., p. 86.
† "Erdbaumechanik," p. 87.

Percentage of mica.	Constant A.
0	200
1	130
5	40
10	18
20	7.5
40	3.5
100	1.0

The values in this tabulation have been plotted in Fig. 4. The steep slope of the curve near the origin of abscissas demonstrates that in the region of low mica content a slight variation in the percentage of flat grains causes a very

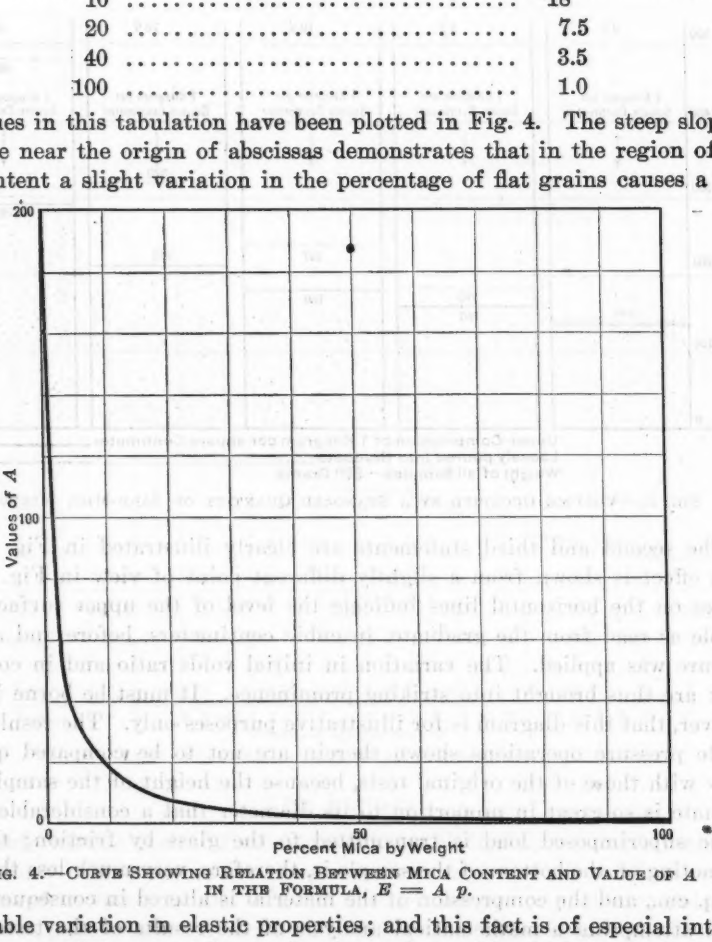


FIG. 4.—CURVE SHOWING RELATION BETWEEN MICA CONTENT AND VALUE OF A IN THE FORMULA, $E = A\ p$.

appreciable variation in elastic properties; and this fact is of especial interest, because most of the natural granular formations with which the engineer has to deal will find their classification somewhere in this region.

DISCUSSION

In reviewing the results furnished by these tests and in correlating them with the science of soil mechanics as a whole, it will be advisable to proceed under three sub-divisions: First, an endeavor will be made to illustrate the importance of tests of this character; second, the most important conclusions to be derived from the present tests will be mentioned; and, finally, the effect these conclusions may have on the general aspect of foundation science and on future methods of investigation and classification of soils, will be examined.

Methods of Investigation.—The Engineering Profession has long recognized the lack of a rational theory of soil behavior, and has attempted in various ways to develop such a theory. Most of the investigations, however, have started with the separation of the soils into their basic constituents, in an endeavor to obtain an idea of the behavior of the mass from a knowledge of the physical and chemical characteristics of the component parts. This procedure does seem a rather obvious one to follow, but there is no analogous method in the investigation of other materials. Steel, for example, has been tested so extensively that it is now reasonably sure that a given specimen will stand a certain unit stress in tension. The chemist and the metallographer are engaged in finding out why this is so, but the structural engineer has never been directly interested in the "why". He is content to know "how much".

Similarly, electrical engineers are accustomed to using copper as a conductor and glass as an insulator, and know within reasonable limits what is to be expected from each material, but they did not arrive at this knowledge by studying the molecular structure of copper and glass; they merely experimented with the materials until they knew how they behaved, without directly questioning why. Now there is no reason why this experimental method of investigation should not be applied to soils. It would be very useful, of course, to understand thoroughly the basic causes of soil behavior; but it is more logical, and of much more immediate interest to the engineer, to determine first what that behavior is. The whole point is that the experimental method of studying the physical characteristics of soils (compressibility, permeability, elasticity, etc.) should be generally adopted not only because it yields results which are of immediate value to the engineer, but also because it is the method which has been successfully followed in the study of all other materials of engineering.

The Compressibility of Soils.—First and foremost, the designer is interested in the settlement of his building. He wants to know how much the total ultimate settlement will be, how long a time will elapse before this final point is reached, and at what rate the settlement will proceed. Now, the settlement of a load imposed on a soil can be due only to two factors: (1) a lateral flow of the soil immediately beneath the foundation toward points of lower pressure; and (2) a change in volume of the underlying soil mass. The motion due to lateral flow can be arrested by surrounding the stressed area with deep sheet-piling. Volume change, however, is a function of the compressibility of the material, and cannot be prevented. It behooves the engineer, therefore, to obtain accurate knowledge not only of the amount of compression he must expect, but also of the rate at which this compression proceeds.

In other words, if a designer is furnished a series of mechanical analysis curves and a schedule of the chemical constituents of the underground on which he intends to build, he is given very little useful information; but if he has a series of pressure-compression and time-compression diagrams, he can proceed intelligently with his design and feel sure that his results are reasonably near the truth. As an illustration, take samples from two deposits corresponding to the mixtures used in these tests, the deposits being alike in every respect except that one contains 1% of mica and the other 5 per cent. An

investigator would probably notice that the second sample included more mica than the first, but could not say with any degree of certainty how much either contained. The chemical properties of the individual grains in the two samples are the same, and the mechanical analysis curves are identical. On the other hand, if the engineer built two identical structures, one on one deposit and one on the other, using a soil pressure of 1 ton per sq. ft., the second building would settle at least four times as much as the first. A single pressure-compression diagram for each sample would have shown at once that the materials were totally different in compressibility, and would have warned the designer that he could not apply the same calculations to both buildings.

Tests at Lateral Confinement.—In order that any given method of testing the characteristic properties of a material may be of value, it must fulfill two requirements: First, it must yield results which are directly applicable to the problem in hand; and, second, it must be simple, accurate, and economical in operation. In the study of the compressibility of soils, both these requirements are met by compression tests on samples which are prevented from expanding laterally.

In the first place, the loaded soil beneath a foundation, to all intents and purposes, is laterally confined; confinement is produced by the weight of the surrounding material, and, when necessary, by an enclosing wall of sheet-piling. A glance at the diagram of a typical loading test will suffice to illustrate this condition. The portion of the curve nearest the origin is concave upward, and this form of curve is typical for laterally confined materials. It is true that the loading test curve reaches a point of inflection and bends downward as load is increased, but this part of the curve does not directly affect ordinary foundation practice, as the superimposed load should always be kept well within the upper region of the diagram in order to avoid the excessive settlements which correspond to the higher loads. A loaded natural deposit is not, of course, as thoroughly confined as a laboratory sample. However, the joints of a steel bridge are certainly not frictionless. Bridges are designed under the assumption of frictionless joints, and it is found that the error is of comparatively small magnitude. Similarly, the results of a lateral confinement test can be applied to the design of a foundation with the assurance that the calculations are reasonably close to actual conditions.

As to the second requirement, lateral confinement tests can now be made very easily and accurately. The equipment and procedure described herein furnish excellent results on granular materials where the time effect is negligible, and the apparatus developed by Professor Terzaghi, and now in use at the Massachusetts Institute of Technology, furnishes complete data on the variation, both with pressure and with time, of the voids ratio of cohesive, water-bearing soils.

Another important advantage of lateral confinement tests is that they are applicable to all types of soils, irrespective of cohesion. Determination of the compressive strength of clay cubes gives certain valuable information, but one cannot test the strength of a cube of cohesionless sand, because such a cube cannot be made. By the lateral confinement method, however, a test can be made on any material, and samples which differ widely in their characteristics can thus be readily compared.

Conclusions from Present Tests.—The outstanding conclusions to be drawn directly from the tests on sand-mica mixtures described herein are obvious from a study of the diagrams and need only to be enumerated. They are: (1) Other things being equal, the initial voids ratio of uncompressed granular masses increases with increasing mica content; and (2), other things being equal, the greater the proportion of flat grains in a granular mass, the greater is the compressibility of the mass. It must be distinctly understood that the term, "compressibility," refers to both positive and negative compression; that is, that the elastic expansion of the mass also increases with increasing mica content. These two basic results postulate other conclusions as to structure, analysis, and classification of soils in general.

Structure of Granular Masses.—In 1899, Professor Slichter* attempted a mathematical analysis of the permeability of soils, and in connection therewith showed that the volume of voids of a mass of equal spheres could range within the limits of 47.6 and 26%, depending on the density of the mass. Ordinary sand is not an accumulation of equal spheres, but it does not vary a great deal from this theoretical condition, the limits of volume of voids of equilibrium structure ranging from about 23 to about 50%, corresponding to voids ratios from 0.3 to 1. These figures indicate that voids ratios greater than 1 are not consistent with equilibrium.

When it was noticed that fine clays were in perfect equilibrium with voids ratios as high as 3, and that the voids ratio of colloidal precipitates could range around 20 and still preserve equilibrium, it became evident that some factor which was negligible in coarse-grained material assumed a prominent rôle in fine-grained sediments. This factor is the zero friction or true cohesion, and a knowledge of the mechanics of zero friction serves to explain fully the possibility of these high voids ratios.†

There is now conclusive evidence that another factor will seriously affect the volume of voids of an accumulation of grains, namely, the presence of scale-like particles. The mica tests show that voids ratios as high as 8 are consistent with equilibrium in masses consisting of coarse-grained constituents, in which the zero friction is quite negligible. In other words, it is necessary to modify present ideas of structure, and to take into account the fact that not only the size of the grains, but also their shape, will materially alter the magnitudes of the limiting voids ratios which are consistent with the equilibrium of the mass.

Mechanics of Compressibility.—It has been stated that the engineer is primarily interested in the "how" of the behavior of structural materials, rather than in the "why". The experimenter, however, who is attempting to formulate a basic theory of behavior, will save himself a great deal of unnecessary work if he has an idea of the causes underlying the characteristics he is investigating. It will be useful, therefore, to visualize the phenomenon of compressibility in the new light afforded by the present tests.

It is a well-known fact that the fine-grained clays are much more compressible, and expand to a greater degree after compression, than the coarse-grained sands. The effect of zero friction cannot be used to explain this action,

* Annual Report of U. S. Geological Survey, 1899, p. 311.

† "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., pp. 64-66.

because friction is a non-conservative force. To express it more simply, assume a structure with a large volume of voids caused by true cohesion between individual grains. It is easy to imagine these intermolecular bonds being broken by external pressure so that the grains can slip farther down in the mass, giving the effect of a highly compressible material. However, it is difficult to conceive of the grains climbing up again on their neighbors after pressure has been removed.

Hypotheses have also been advanced which ascribe the characteristic properties of clays to the presence of a "clay-humus complex", or to the colloidal material present in the clay,* but an explanation of this type should be accepted only after all other possibilities have been exhausted. After all colloidal particles are merely particles of small size, and there should be nothing essentially mysterious in their behavior. At any rate, if it is possible to reproduce the compression characteristics of a colloidal material in a mass which contains absolutely no trace of colloids, it is rather unwarranted to ascribe the compressibility of the fine-grained sample to the presence of colloidal particles.

This reproduction has actually been accomplished with the sand-mica mixtures. The scale-like grains, acting as tiny beams and columns, are capable of undergoing considerable elastic deformation without appreciable volume change, with the result that the entire mass becomes more and more compressible as the number of these flat grains is increased. It is necessary, therefore, to accord the flat grains a place of major consideration in ideas of soil behavior, and to realize that they are chiefly responsible for the high compressibility common to fine silts, quicksands, and clays.

Analysis of Soils.—The preceding considerations justify the statement that any system of analysis of soils which neglects the presence of flat grains in the mass can yield no reliable indication of the properties of the material. At present, there is no known method of determining the mica content of a soil sample. By the use of a microscope one can see the mica grains in a quicksand, but as to how many there are, and what proportion their weight bears to the weight of the total mass, one cannot even hazard a guess. It is known that these mica grains constitute an extremely important factor. In the region of low mica content a slight variation in percentage of flat grains will cause a very appreciable variation in compressibility. It seems illogical, therefore, to proceed with elaborate and tedious methods of mechanical and microscopic analysis, and with still more elaborate and expensive determinations of the colloidal content, at the same time blithely ignoring a factor of major importance simply because there is no way of taking it into account.

As mica content cannot be determined directly, one must use indirect methods; and as engineers are not primarily interested in the value of the mica content itself, but rather in its effect on the compressibility of the material, the logical thing to do is to test the compressibility directly. This comes back to the argument advanced in the first part of this paper, and shows once more why real and lasting results can be expected only by trying to build up a theory of the behavior of soils on the basis of a thorough experimental investigation of their physical characteristics.

* See the Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, etc., *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 884.

Classification of Soils.—As a direct sequence to the foregoing discussion it may be stated that any system of soil classification which neglects the presence and the effect of scale-like particles in the soil is incomplete and erroneous. Inasmuch as the exact quantity of mica in a soil sample cannot be determined, there is no justification for classifying soils on the basis of the grain size of the constituents, nor in reference to their content of particles below a certain arbitrary grain diameter.

Engineers are accustomed to distinguish between steels by noting their relative tensile and compressive strengths, and not by referring to their crystalline or molecular structure. Similarly, they should attempt to express the characteristics of soils by referring to the physical behavior of the material under stress. On account of the complexity of the problems of soil mechanics, the formulation of a complete system of this kind will be possible only after many years of research; but they should always keep the ultimate end in view, and plan investigations in such a manner as to proceed logically toward this end.

In the meantime, they may use certain devices which will enable them to compensate for lack of exact mathematical expressions. An interesting example of this type arises from the comparison of the compression curves of natural soils with those of sand-mica mixtures, as shown in Fig. 4. The heavy curves in the lower part are pressure-compression diagrams for various natural soil samples. Above each diagram is shown in dotted lines the diagram of the sand-mica mixture which corresponds most closely to that of the natural soil. The striking resemblance between the soil curves and the sand-mica curves indicates that a comparative classification of compressibility is not at all unreasonable. If every soil investigator had a complete set of standard sand-mica curves, including not only those resulting from the present tests (0, 1, 5, 10, 20, 40, and 100% mica), but also those for intermediate mica contents, the compression diagram for any clay could be completely described by mentioning one voids ratio and referring to one of the standard curves. As a matter of fact, the Bureau of Public Roads of the United States Department of Agriculture, in co-operation with the Massachusetts Institute of Technology, is engaged in the development of a system of soil classification on the basis of a comparative system of this type.

The only essential difference between the clay curve and the corresponding sand-mica diagram is at the extreme left end of the rebound curve, where the coarse-grained material is seen to expand more than the clay. This fact is immediately explainable on the basis of intermolecular action, which has no effect on the sand or mica grains, but produces a true cohesion between the fine clay grains, preventing the mass from expanding to the limit of its elastic energy.

Supplementary Remarks.—In connection with further research on compressibility it will be extremely desirable to develop a more accurate mathematical analysis of compression curves in general. If it is true that criticism is justified only when accompanied by suggestions for improvement, this paragraph should be entirely omitted; for the writer, after many vain attempts to

formulate more exact equations, was forced to return to Terzaghi's original analysis.*

During these attempts, however, curves of moduli of elasticity were plotted against pressure. These curves showed that for the more compressible materials there is a very considerable discrepancy between Terzaghi's simple logarithmic curve and the actual expansion curve. Terzaghi attempts to account for the discrepancy by referring to successive accumulations and releases of unbalanced frictional stresses, but there are certain outstanding features, such as the extremely high value of the modulus of elasticity at the beginning of the rebound curve, which are so uniformly characteristic for all types of soils that they must eventually yield to analysis from a quantitative standpoint. The ideal condition would be to express completely the compression characteristics of a material by one, or, at most, by two or three, numerical constants; and the successful formulation of an exact mathematical analysis of compression curves would unquestionably be an important step toward a thorough understanding of the mechanics of compressibility of soils.

Summary and Conclusions

This paper presents an outline of a series of preliminary tests made on the compressibility of mixtures of dune sand and white mica (grain size, 0.417 to 0.589 mm.), at lateral confinement. The test data furnished curves of voids ratio against intensity of pressure for each of the samples of varying mica content. From these curves the following specific conclusions may be drawn:

1.—Other things being equal, the initial voids ratio of compacted, uncompressed granular masses increases with increasing mica content.

2.—Other things being equal, the greater the proportion of flat grains in a granular mass, the greater is the compressibility of the mass, and the more marked is its elastic expansion after removal of load.

These facts indicate the following general conclusions:

3.—The content of flat grains in a soil has a marked effect on the magnitude of the limiting voids ratios consistent with equilibrium.

4.—The compressibility of typical fine-grained sediments is not ascribable to the small size of the grains nor to the presence of colloidal particles.

5.—Any system of soil analysis which fails to determine the percentage of flat grains in a sample can yield no uniformly valid results.

6.—Any system of soil classification which neglects the presence and the effect of scale-like particles is incomplete and erroneous.

7.—The study of foundation problems by investigation of the physics and mechanics of soils is the only method which may be expected to yield results of present utility and of ultimate value to the Engineering Profession.

* "Erdbaumechanik," p. 86 *et seq.*

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PAPERS AND DISCUSSIONS

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SILTING OF THE LAKE AT AUSTIN, TEXAS

By T. U. Taylor,* M. Am. Soc. C. E.

In 1890, cross-sections of the old lake formed by the Austin Dam were taken at different points, as shown in Table 1 and in Fig. 1. The crest of the old Austin Dam was 60 ft. above the toe, or apron of the down-stream face, whereas the overflow crest of the new dam is 9 ft. lower, or 51 ft above the toe.

The water first flowed over the crest of the old dam in May, 1893, and there were at this time 79 537 303 cu. yd., or 49 300 acre-ft. of water in the lake, rising to the level of the crest of the old dam. In February, 1900, there were 41 528 810 cu. yd., or 25 741 acre-ft. Thus, the water capacity of the lake of February, 1900, was only 52% of the capacity of that of May, 1893. If these volumes are estimated in depths on a square-mile base, the result is, in 1893, a volume of water equal to a depth of 78.6 ft. and, in 1900, to a depth of 40.2 ft. of water and 38.4 ft. of silt. This was the result from May, 1893, to January, 1900, a period of 6.75 years.

Table 1 shows the maximum and mean depths of water for 1893 and 1900; the maximum and mean depths of silt for 1900; and the percentage of silting at the respective stations.

The maximum depth of silt is not always equal to the differences of the maximum depths of water for 1893 and 1900 because the silt has shifted at several points. This was very noticeable at Station 12.525, known as Santa Monica, or Sulphur Springs. The column of percentages is the ratio that the cross-section of 1900 (of silt) bore to the original cross-section of water. Thus, at Santa Monica Springs (Station 12.525), 78.9% of the original cross-section has been filled in 6.75 years.

Between Stations 0 and 17.75, the silt in 1900 was a fine, impalpable, absolutely gritless deposit. From Station 17.75 to the head of the lake, there was a distance of nearly 3 miles where the silt consisted largely of sand.

Note.—Written discussion on this paper will be closed in **August, 1928.**

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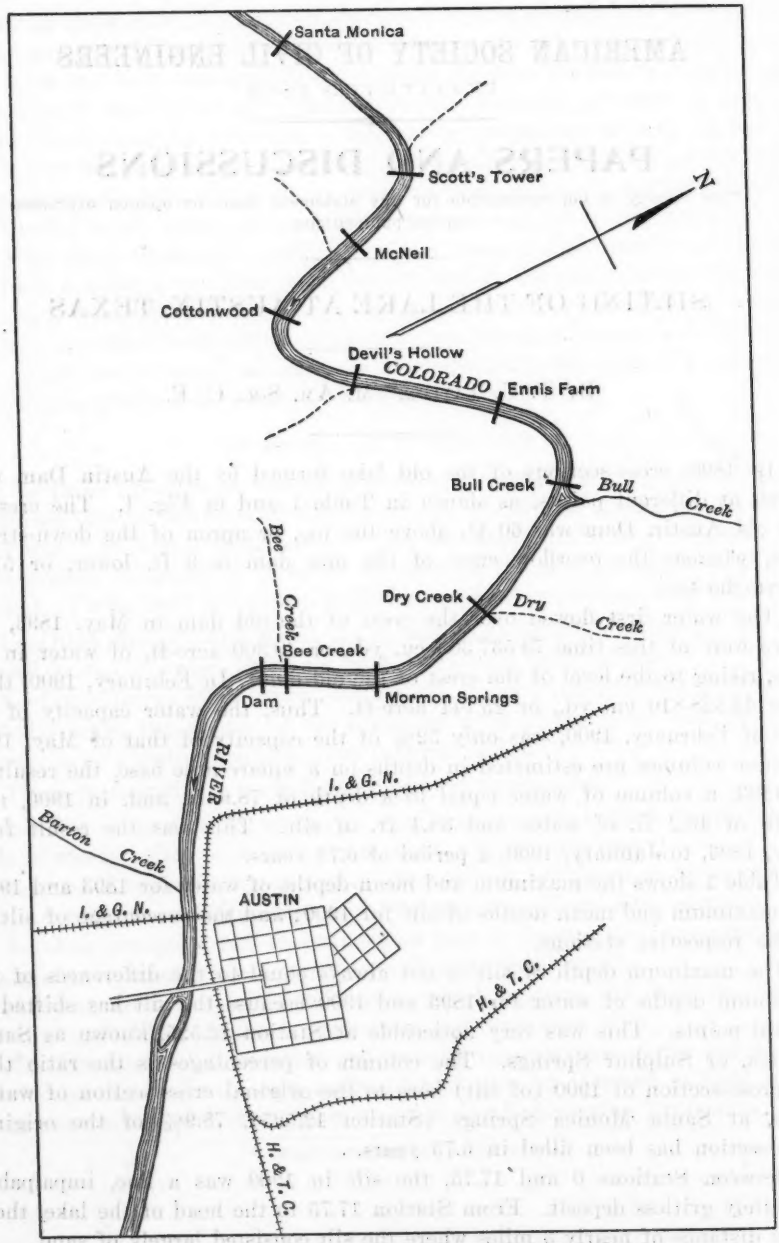

FIG. 1.—MAP SHOWING LOCATION OF AUSTIN DAM.

TABLE 1.—DEPTHS OF WATER AND SILT IN THE RESERVOIR FORMED BY THE OLD DAM AT AUSTIN, TEXAS.

MAXIMUM DEPTH.

Distance of station, in miles, from dam.	DEPTH OF WATER, IN FEET.		DEPTH OF SILT, IN FEET.
	1893.	1900.	1900.
0.0	66.0	39.0	27.0
0.221	65.7	38.0	27.7
1.173	63.8	35.5	28.3
2.698	56.0	31.5	24.5
4.012	47.8	31.5	19.0
5.337	47.5	27.5	23.0
6.799	47.0	22.0	26.0
7.810	44.8	20.5	25.0
9.189	40.4	13.4	29.0
10.221	40.9	13.5	27.4
12.525	29.4	9.8	26.0
13.425	24.0	12.5	16.0
14.725	16.6	12.0	15.0
16.225	13.2	9.5	7.0
17.725	3.7	2.2	1.5

MEAN DEPTH.

Distance of station, in miles, from dam.	MEAN DEPTH OF WATER, IN FEET.		MEAN DEPTH OF SILT, IN FEET.	Amount of fill, percentage of total.
	1893.	1900.	1900.	
0.0	40.1	27.5	12.6	31.2
0.221	37.6	26.5	11.1	32.2
1.173	30.6	21.7	8.9	20.2
2.698	33.4	21.5	11.9	38.8
4.102	38.0	23.5	14.5	37.9
5.337	36.7	23.0	13.7	42.0
6.799	36.3	17.4	18.9	56.3
7.810	30.7	15.4	15.3	56.3
9.189	30.8	10.2	20.6	66.9
10.221	27.2	7.9	19.3	71.8
12.525	20.3	5.5	14.8	78.9
13.425	17.2	9.0	8.2	50.4
14.725	13.2	10.0	3.2	60.0
16.225	11.2	8.8	2.4	30.0
17.725	5.6	3.9	1.7	33.0

The Pedernales River traverses sandy valleys and enters the Colorado River several miles above the head of the old lake, and at flood stages it is heavily charged with sand. When these flood waters strike the relatively still waters of the lake and the velocity is checked, this heavy sand is deposited. At occasional points below the head of the lake, small bars of sand were found near the mouths of small canyons or creeks. The cross-sections of the old lake are shown in Fig. 2. The vertical scale is ten times the horizontal. (See also Table 2).

From March 15 to April 17, 1899, the water level of the lake was a little more than 10 ft. below the crest of the dam. The water again commenced to overflow on April 21, 1899, when the river rose to a height of 9.8 ft. above the old crest. This flow continued until June 2, 1899, and its result on the cross-sections near the head of the lake was marked. The sections at Stations

13.425, 14.725, 16.225, and 17.725 were scoured out from 2 to 3 ft. deeper than the sections of 1897, and, at Station 14.725, a sand-bar was deposited on the inside (left) of the curve of the river, which contracted the channel to less than one-half its former width.

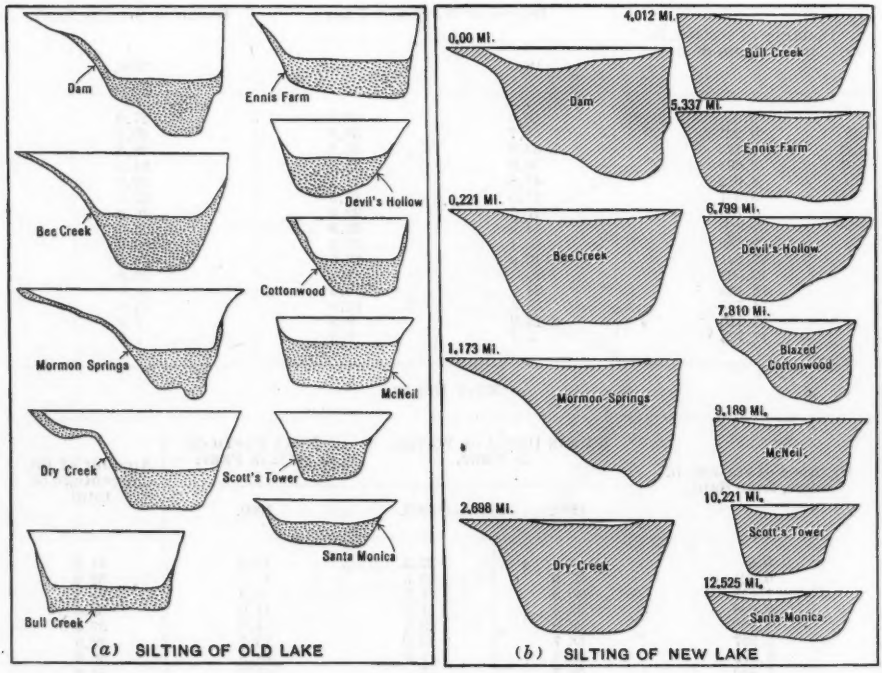

FIG. 2.

From above Colorado City, on the Texas and Pacific Railroad, the river flows through a hilly country for 300 miles, and absorbs in its course the waters of the Conchos, the San Saba, the Llano, and Pecan Bayou, as well as those of the Pedernales. All the country drained by these tributaries is hilly, with the exception of a few miles along the head of the Colorado and the Conchos Rivers.

SILTING OF NEW LAKE AUSTIN

The contract for the present Austin Dam was let in 1911, and the dam was partly completed by the summer of 1913. In September of the same year, the lake was filled with water up to the present spillway, which is 51 ft. above the toe of the old dam. This datum was the one usually adopted. The dam has not yet been accepted by the City of Austin on account of litigation. In 1918, Mr. Guy A. Collett, the Receiver, with the consent of the Federal authorities, sold the water in the lake to the rice growers in the lower counties, for the purpose of irrigating the thousands of acres of rice on both sides of the Colorado River. The new dam is shown in Fig. 3, which is

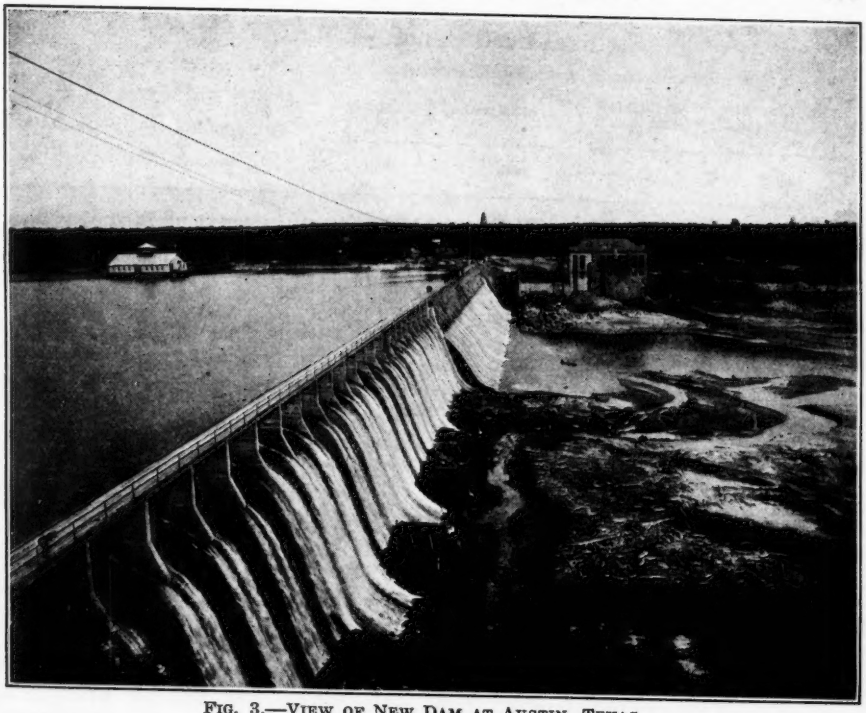

FIG. 3.—VIEW OF NEW DAM AT AUSTIN, TEXAS.

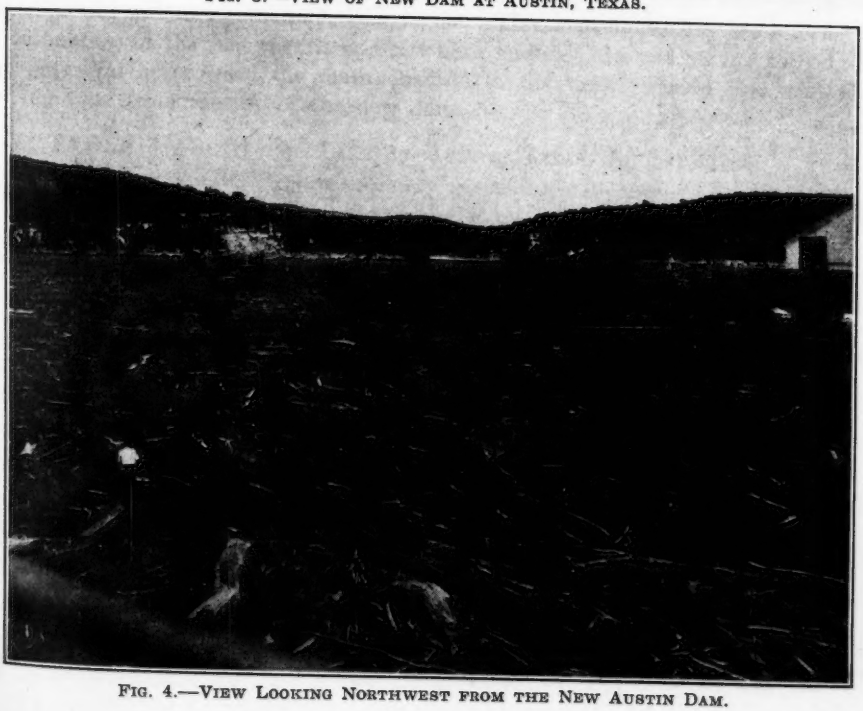

FIG. 4.—VIEW LOOKING NORTHWEST FROM THE NEW AUSTIN DAM.

TABLE 2.—SILT IN OLD LAKE.

Station.	Name.	AREA, IN SQUARE FEET.		VOLUME OF WATER, IN ACRE-FEET, BELOW CREST OF DAM.	
		1893.	1900.	1893.	1900.
0.0	Dam	48 000	33 000	1 286	878
0.221	Bee	48 000	32 500	5 100	3 504
1.173	Mormon	40 400	28 200	7 316	4 796
4.012	Bull	32 980	20 480	5 419	3 250
5.337	Ennis	34 500	20 000	5 531	2 853
6.799	Devils	27 925	12 200	3 010	1 315
7.810	Cottonwood	21 200	9 265	3 727	1 422
9.189	McNeill	23 400	7 750	2 519	782
10.221	Scotts	16 870	4 750	4 757	1 170
12.525	Santa Monica	17 200	3 630	.....	.....
Sub-total	.....	.....	.....	44 377	23 488
13.425	.....	11 180	5 500	1 548	498
14.725	.....	7 700	3 300	1 487	670
16.225	.....	5 100	3 500	1 163	591
17.725	.....	2 880	1 940	725	494
Grand total	.....	.....	.....	49 300	25 741

a view from the west bluffs looking east. The house on the left was a boat-house under which launches, large enough for twenty people, could pass, but by 1924 the silt had reduced the depth to such an extent that small canoes could not navigate under the house. This boat-house has been torn down because of the fact that it was left high and dry, the silt having forced the water farther toward the main channel to the west. Fig. 4 is a view looking west from the top of the new dam.

TABLE 3.—SURVEY OF LAKE AT AUSTIN, TEXAS, AUGUST, 1926.

Distance of station from dam, in miles.	WATER AREAS, IN SQUARE FEET, BELOW CREST OF DAM.			WATER VOLUME, IN ACRE-FEET, BELOW CREST OF DAM.			
	1913.	1922.	1924.	1913.	1922.	1924.	1926.
0.0	36 000	13 000	11 600	981	322	249	97
0.221	37 300	11 100	6 980	3 790	982	520	224
1.173	28 400	5 930	2 040	5 037	920	316	143
2.698	26 100	3 030	1 380	4 000	470	224	86
4.012	24 130	2 870	1 430	4 009	469	227	108
5.337	25 800	2 970	1 390	4 075	582	292	142
6.799	20 200	3 600	1 900	2 450	357	250	161
7.810	19 800	2 230	2 186	3 000	382	308	189
9.189	16 100	2 350	1 460	1 695	310	187	88
10.231	11 000	2 610	1 530	2 988	568	328	239
12.525	10 400	1 460	820	.....	....	....	....
Total volume, in acre-feet	.....	.....	.....	32 025	5 362	2 901	1 477

During the latter part of the summers of 1922, 1924, and 1926, the writer made measurements of the cross-sections of the present lake at the old

stations, and found the results to be as given in Table 3, and as illustrated by the cross-section in Fig. 2(*b*).

Theory of Silting of Lake Austin

The silting of reservoirs for purposes of comparison is best reduced to heights or depths on a square-mile base. To derive an expression for the quantity of silt deposited in a given time, let x = the depth, in feet, of silt deposited by each foot in height per year; and h = the original depth of water in the reservoir. The depths of water at the end of 1, 2, 3, and n years are:

At the end of 1 year, depth of water = $h\ (1-x)$
At the end of 2 years, depth of water = $h\ (1-x)^2$
At the end of 3 years, depth of water = $h\ (1-x)^3$
At the end of n years, depth of water = $h\ (1-x)^n$

Hence, if d = the depth of water at the end of n years,

$$d = h\ (1-x)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This will not be true for any reservoir in which there is an appreciable current acting on the bottom of the basin, that is, on the upper surface of the silt. In the old lake at Austin, the lake level could sink to 10 ft. below the crest of the dam, and still have a fair current in the penstocks, as the bottom of the forebay was 12 ft. below the crest of the old dam.

The measurements on the new lake were carried no farther up the lake, for the reason that the next station above Santa Monica was filled entirely with sand. In fact, sand was encountered about ½ mile below Santa Monica, or at a point 12 miles from the dam. Table 3 includes only the fine gritless mud. The volume of water in:

1913 = 32 029 acre-ft.
1922 = 5 362 acre-ft.
1924 = 2 901 acre-ft.
1926 = 1 477 acre-ft.

If these are reduced to the depths, in feet, on a square-mile base, the volume of water in:

1913 = 50.05 ft.
1922 = 8.38 ft.
1924 = 4.53 ft.
1926 = 2.31 ft.

$$\text{Ratio of water volumes} = \frac{\text{Volume of 1926}}{\text{Volume of 1913}} = 0.0461 = 4.61\%$$

Thus, in the thirteen years the Austin Lake has filled 95.39% of its volume in 1913. The water has now (1926) 4.61% of the volume it had in 1913.

In Equation (1), $h = 50.05$, and the depth of water = 2.31. Hence,

$$50.05\ (1-x)^{11} = 2.31$$

and,

$$x = 0.20$$

That is, on an equal base, an average of 20% of silt was deposited each year. Fig. 5 illustrates the relative quantity of silt and water in September, 1926.

Drift

The Colorado River of Texas, with its tributaries, traverses nearly 2 000 miles of wooded territory, and every rise brings its sinuous ribbon of drift, which ranges in width from 40 to 200 ft. When the first drift strikes the gates or buttress walls of the Austin Dam, it spreads out over the entire width of the lake, and banks up behind, until the complete surface of that part near the dam is covered with drift from shore to shore. At times, this covers an area more than ¼ mile wide and ½ mile long.

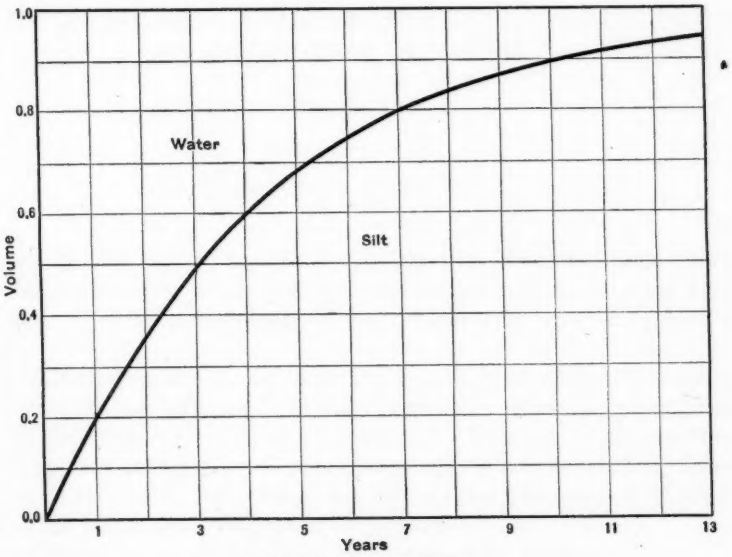

Fig. 5.—Silt Curve.

Fig. 4 was taken from the top of the dam, looking northwest. A line, joining the man in the foreground with the building on the right, points straight up the river. This drift sometimes covers an area of 80 acres, and the velocity of the water in the lake gradually compacts the drift toward the dam, making it more and more dense.

As the water recedes and the lake level sinks, the part of the drift between the man and the power-house rests on the alluvial soil beneath—the result being that the silt near the dam is in reality mud reinforced with sticks, débris, and all manner of floating logs.

In the background is noticed a strip of driftless water. This water is in Bee Creek, above which the silt is almost free from drift, because the velocity of the water in the lake compresses the drift area into that section between the creek and the dam.

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PAPERS AND DISCUSSIONS

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THE EYE-BAR CABLE SUSPENSION BRIDGE AT FLORIANOPOLIS, BRAZIL

Discussion*

By Hardy Cross, M. Am. Soc. C. E.

Hardy Cross,† M. Am. Soc. C. E. (by letter).‡—That this paper represents a valuable contribution to the art of bridge design and construction is at once recognized. The writer's comments are restricted to only one aspect of the design.

The authors indicate doubt as to the propriety of applying to suspension bridges criteria for stiffness borrowed from other types of bridge construction. In this they are probably correct. The question is of importance in considering the economics of continuous and other structures. Live load deflection is often only the obvious and measurable phenomenon which accompanies other phenomena of greater importance.

Large deflection of bridges may be objectionable because:

(*a*) It produces unsightly distortions. This can almost always be corrected by camber.

(*b*) Distortion produces grades in the roadway which interfere with traffic operation. This is a definite limit on deflection; so far as the writer knows, it occurs only in suspension bridges.

(*c*) Vibration accompanies deflection, and vibration is to be avoided because:

(1) It wears out the structure.
(2) It may be psychologically objectionable, producing a sense of fear in the pedestrian or passenger.

In most bridges the objections to deflection are associated with vibration. It is clearly recognized that vibration wears out—or tends to wear out—a

* Discussion on the paper by D. B. Steinman and William G. Grove, Members, Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Prof. of Structural Eng., Univ. of Illinois, Urbana, Ill.

‡ Received by the Secretary, December 5, 1927.

structure; it loosens rivets in steel work and probably destroys bond in reinforced concrete. It may also produce what is usually called fatigue. While this is clearly recognized qualitatively, a quantitative measure seems impossible in the present state of the structural art. Structural mechanics commonly ignores entirely those phenomena which occur while parts of the structure are in motion and restricts itself to conditions existing when the structure has come to rest. Vaguely, designers treat the effects of deflection by the application of an "impact" coefficient.

It is probably true that a large deflection is an indication of high "impact" effects; but that it is a satisfactory measure, or that the durability or the freedom from shock is a function chiefly of deflection, independent of the type of structure, there seems to be no evidence at all.

As regards the objectionable psychological effect of vibration, it is probable that deflection is no fair measure at all. Objectionable movement probably means high acceleration; passengers apparently are not conscious of uniform movement.

It is possible to establish certain relations between the acceleration produced at any point by a unit load at any other point and the velocity of the load, but this does not help the problem very much.

The whole subject becomes of great importance in many cases. The limitations on deflection borrowed from practice in the design of simple spans affect the economy of continuous girders and trusses adversely. There is not much basis for a comparison of the types. The writer would appreciate any information as to who first established the existing rules in this regard, and why they were established.

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STATE RECLAMATION IN WASHINGTON

Discussion*

By R. K. Tiffany, M. Am. Soc. C. E.†

R. K. Tiffany,‡ M. Am. Soc. C. E. (by letter).§—The discussion of State reclamation following the writer's paper has been interesting, although it has all tended more or less toward the conclusion that State reclamation, as such, is decidedly a questionable undertaking for the present time. Mr. Luper's suggestion‖ that the State might aid reclamation by the issuance of State bonds with a limited liability, is worthy of consideration, if and when the situation becomes such that the State, in order to secure agricultural development, is justified in loaning its credit. The writer did not criticize the Washington State Law providing for certification of irrigation district bonds, simply for the reason that it has not been sufficiently used to give basis for judgment as to its value.

Mr. Swendsen¶ questions the advisability of reclaiming any unoccupied areas until existing projects are settled, and Mr. Stevens credits the writer with a like opinion. The writer agrees with Mr. Stevens¶ rather than with Mr. Swendsen on this point to this extent, that he believes there are certain localities and certain conditions under which new, or extensions of existing projects, should be made, even at the present time. Washington, in the opinion of Professor Waller,** presents, or will in the future present, such opportunities for profitable development owing to the relatively small area of irrigated land in the State and to the rapid growth (past and prospective) of her industry and commerce.

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* Discussion on the paper by R. K. Tiffany, M. Am. Soc. C. E., continued from September, 1927, *Proceedings*.

† Author's closure.

‡ Superv. of Hydraulics, State Dept. of Conservation and Development, Olympia, Wash.

§ Received by the Secretary, December 19, 1927.

‖ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1359.

¶ *Loc. cit.*, p. 1361.

** *Loc. cit.*; p. 1362.

The points brought out by Messrs. Jacobs and Newell have to do largely with the broader National aspects of reclamation, rather than the question of State reclamation. With Mr. Jacobs the writer will agree to the extent that State reclamation should not be condemned and abandoned by reason of the errors and losses that have occurred in Washington.* The experience in these first efforts at State reclamation may be useful to Washington and to other States when the time comes that there is need for the States to take up actively the problem of increasing their agricultural production.

As to the National reclamation policy, Mr. Newell from his broad experience points out many errors in the past and need for a more sound and conservative policy in future development.† In the writer's opinion there is still much to be said in support of the doctrine that the Federal Government should continue to aid in development of the Arid West.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1363.
† *Loc. cit.*, September, 1927, Papers and Discussions, p. 1753.

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PAPERS AND DISCUSSIONS

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RELATION OF ROAD TYPE TO TIRE WEAR

Discussion*

By O. L. Waller and H. E. Phelps, Members, Am. Soc. C. E.†

O. L. Waller‡ and H. E. Phelps,§ Members, Am. Soc. C. E. (by letter).‖—Considerable study has been made by different investigators on the wear of automobile tires due to different types of roads. It was the writers' aim to bring this material together, and to discuss some features of it, but there was no thought of laying down working rules. The writers were not unfair in their use of data, but simply presented the data gathered by others and drew general conclusions therefrom that are believed to be valid although necessarily approximate.

The writers had little to do with the selection of any of the pavements that were tested, but believe that at least the tests made by the State College of Washington were run over pavements fairly representative of the several road surfaces. The lack of sufficient money to make a more extended series of tests does not discredit those that were made. The fact that some types of pavements were not tested at all, is not unfair to those types, nor is it unfair to fail to find a perfect bitulithic pavement. In the Pacific Northwest it is very difficult to find any considerable length of asphaltic pavement in good condition since very little has been constructed on State highways in recent years. The writers are glad to note Mr. Carmichael's statement¶ that the bitulithic pavement tested was in good condition and they believe the index number for average bitulithic, given in Table 3,** confirms that fact. This index number is very favorable to bitulithic, and not the reverse, as Mr. Carmichael seems to have found it.

* Discussion on the paper by O. L. Waller and H. E. Phelps, Members, Am. Soc. C. E., continued from November, 1927, *Proceedings*.

† Authors' closure.

‡ Vice-Pres., State Coll. of Washington and Head, Dept. of Civ. Eng., Pullman, Wash.

§ Prof., Highway Eng., State Coll. of Washington, Pullman, Wash.

‖ Received by the Secretary, December 21, 1927.

¶ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2364.

** *Loc. cit.*, August, 1927, Papers and Discussions, p. 1194.

The ideal conditions under which tire wear tests should be conducted were pointed out by the writers,* and these conditions are substantially re-stated by Mr. Carmichael,† but without any suggestion as to how or where the ideal can be found. It would be interesting to know the location of two parallel or adjoining roads under practically the same climatic, grade, and age conditions, one paved with concrete and one with bitulithic, and both surfaces nearly perfect and at least five years old. The State of Washington has used concrete exclusively for pavement for more than eight years and there are no ideal bituminous pavements in the State highway system, nor is there any long stretch of ideal macadam or broken rock surface road.

Of course, a test in which the tires are run to destruction is better than a short-time test. The experiment should be extensive enough to eliminate the effect of variations in the quality of individual tires. An average of the extreme variations in the condition of each type of road surface should be determined in such a way that the resulting index number would accurately reflect the tire-wearing quality of that surface and the resistance of any particular type of tires manufactured. Such a test would be very expensive. It is believed that the data already secured have been thoroughly studied by careful investigators and that the results secured are reliable.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1202.
† *Loc. cit.*, November, 1927, Papers and Discussions, p. 2365.

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THE LAKE WASHINGTON SHIP CANAL, WASHINGTON

Discussion*

By W. J. Barden and A. W. Sargent, Members, Am. Soc. C. E.†

W. J. Barden‡ and A. W. Sargent,§ Members, Am. Soc. C. E. (by letter).‖—The authors are appreciative of the information given and comments made in the discussions of their paper.

Mr. Hussey¶ thinks that more direct and positive provision should have been made for the relief of possible hydrostatic pressure under the lock floor, and suggests the use of valves or a system of drains. Both these methods have been used in some instances, but have not always been found entirely satisfactory. The present practice, where the floors are not designed to resist full upward pressure, appears to be to use a paving of concrete blocks with weep-holes. In the Seattle Lock, having in mind the more or less impervious character of the foundation, the cut-off walls provided, and the clay back-fill which was carried to a depth of 20 to 30 ft. over the steps on the back of the walls, the designers evidently considered that hydrostatic pressure sufficient to endanger the lock floor was unlikely to develop and the results have apparently justified their conclusion. It may be added that should failure occur in the future, repairs would not be difficult, and sufficient provision against subsequent trouble could probably be made by drilling a system of weep-holes.

The suggestion that a flow of fresh water conducted along the timbers of the guard-gate and miter-sill would have protected them from marine borers is of much interest. It is possible that such a procedure might be effective in keeping the steel gates free from barnacles which adhere to and destroy the paint, necessitating extensive cleaning and repainting every year.

Approval has recently been secured to deepen and widen the present salt-water basin and to lower the intake of the salt-water drains 7 ft., and this work will soon be undertaken. It is believed that this will materially reduce the quantity of salt water passing into the lakes. The latest salinity tests taken at the end of the dry season show a little less salt than usual at such times, due probably to an unusually cool summer with resultant smaller evaporation, thus allowing more water to be wasted through the locks.

* Discussion on the paper by W. J. Barden and A. W. Sargent, Members, Am. Soc. C. E., continued from January, 1928, *Proceedings*.

† Authors' closure.

‡ Col., Corps of Engrs., U. S. A., New York, N. Y.

§ Civ. Engr., U. S. Engr. Office, Govt. Locks, Seattle, Wash.

‖ Received by the Secretary, January 5, 1927.

¶ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 2079.

Mr. Fowler* gives interesting additional historical data and details on canal construction. The following costs may be of interest: The cost of cement f. o. b. cars at the lock site was $1.49 per bbl.; unloading, 2 to 4 cents per bbl.; testing, 5 cents per bbl.; sand and gravel f. o. b. barge at pit, 23.8 cents per cu. yd.; towing, 12 to 18 cents per cu. yd.; unloading and delivering to bunkers at mixer plant, 5 to 8 cents per cu. yd. These costs do not include plant costs or overhead.

Mr. Clapp† also gives some interesting historical data and a description of previous studies leading to the adoption of the present plan. Limitation of space prevented the authors from giving more details concerning these features. The statement that the diversion of the waters of the White River to the Puyallup has a greater beneficial effect in the prevention of floods in the Duwamish Valley than the construction of the canal, is correct. It was intended to list this feature merely as a collateral benefit.

The authors agree with Mr. Sabin's comment‡ that,

"In lock design there are several considerations that render it impracticable to predetermine accurately by theory the action of a certain proposed arrangement of culverts, valves, and ports or laterals".

His analysis of the designs of the Pedro Miguel Lock of the Panama Canal, the latest locks at St. Mary's Falls Canal, and the Lake Washington Lock, so far as these features are concerned, is of great interest, but his statement as to the experience in operating the locks at the two former places would seem to indicate that no materially better results were secured by the designs adopted there than at the Lake Washington Locks. At the St. Mary's Canal Locks, where the culverts are under the lock floor with a system of ports distributed over the floor area, it appears that fluctuations of level from one end of the lock chamber to the other occur with a maximum difference of about 1 ft. While the lock can be filled in about 6 min. by opening all valves simultaneously, in order to reduce this surge, about 10 min. are consumed in ordinary operation. At the Lake Washington Lock, culverts and laterals were designed to fill the lock in 8 min., but the actual time is only about 7½ min. There are no noticeable cross-currents and the up-stream current is not perceptible until the chamber is practically filled and it is then insufficient to affect ships. The latter are always moored to the north wall and the opening of the north valve is started a few seconds before that of the south valve, so as to avoid the possibility of any cross-currents that would tend to pull the ship away from the wall.

At the Pedro Miguel Lock, where the main culverts discharge into small laterals under the floor, the water apparently passes the first two or three ports in the laterals. The discharge is greater near the wall opposite the culverts, and this results in approximately the same distribution as with the laterals or ports in the side walls discharging directly into the chamber. In general, it would appear that whatever the design may be, the most satisfactory results in operation can only be secured as the result of experience and the exercise of good judgment on the part of the operating force.

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 2081.
† *Loc. cit.*, December, 1927, Papers and Discussions, p. 2729.
‡ *Loc. cit.*, January, 1928, Papers and Discussions, p. 273.

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PRECISE WEIR MEASUREMENTS

Discussion*

By A. Streiff, M. Am. Soc. C. E.

A. Streiff,† M. Am. Soc. C. E. (by letter).‡—These experiments confirm Professor Schoder's well-established reputation in the field of experimental hydraulics. They constitute an important contribution to the art of measuring water. Ever since Trautwine in 1889 published a translation of Bazin's experiments, Bazin's formula has been widely used in this country, and hence experiments tending to prove its somewhat large results are of importance.

As Professor Schoder so conclusively shows, the distribution of the velocity of approach is of vital importance in a weir, and it is, therefore, not easy to determine whether Bazin's or Francis, Fteley and Stearns, or Rehbock's formula is the correct one. Not to be overlooked is the great difficulty involved in calibrating a large measuring basin properly, such as a lock with more or less rough walls, as used by Francis, or a section of the Sudbury conduits, as used by Fteley and Stearns. It is easy to obtain an error of 1 or 2 per cent. Professor Rehbock's experiments should be placed in a different class. These are "indoor" experiments, conducted on such a small scale that this latter circumstance may easily introduce results that are 1% or 2% too small. The writer is well acquainted with the hydraulic laboratory of Professor Rehbock, and believes that the measurements were very accurate; but the small size of the flume no doubt caused the experiments to give somewhat small results, if the formula was applied to the measurement of larger discharges.

It is of interest to quote the experiments made at the Amsteg Power House of the Swiss Federal Railways, recently published.§ The results are given in Table 59.

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* Discussion on the paper by Ernest W. Schoder, M. Am. Soc. C. E., and the late Kenneth B. Turner, Esq., continued from January, 1928, *Proceedings*.

† Cons. Engr., Jackson, Mich.

‡ Received by the Secretary, November 1, 1927.

§ "Essais comparatifs dans les canaux de fuite de l'usine d'Amsteg," Departement fédéral de l'interieur, Berne, 1926.

Seemingly, Table 59 would show the greater accuracy of the Rehbock formula over those of Frese or Bazin, especially since the volumetric measurement is closely checked by the titration method and the current meter. Nevertheless, the table shows the tendency of the Rehbock formula to be on the small side, and the Swiss "S. I. A." ("Schweizerische Ingenieur und Architekten Verein") formula is shown to check more closely than that of Rehbock.

TABLE 59.—RESULTS OF EXPERIMENTS AT AMSTEG POWER HOUSE.

Discharge, in volumetric liters per second.	PERCENTAGE ERROR OF WEIR.		
	Frese.	Bazin.	Rehbock.
1 182	+ 1.7	+ 1.5	+ 0.2
2 128	+ 1.9	+ 1.4	− 0.1
3 204	+ 2.5	+ 1.9	+ 0.2
4 371	+ 1.5	+ 0.7	− 0.8

More comparable with the Francis, Fteley and Stearns, and Schoder experiments than the Rehbock formula are the Frese experiments. In Hanover, Germany, Professor Frese (like Francis) made "full-sized" tests in a lock chamber,* which are practically unknown in this country, and are, for instance, not mentioned by R. E. Horton, M. Am. Soc. C. E., in the well-known *Water Supply Paper No. 200.* It happens that the Frese formula also gives results tending to be on the large side, and, hence, excluding Rehbock's formula, there are also the following formulas based on large-scale experiments: Francis, Fteley and Stearns, Schoder, on the small side, and Bazin and Frese, giving larger discharges.

It is of importance that Professor Schoder has so conclusively demonstrated that the weir is a very sensitive contrivance. On the Continent of Europe the inherent instability of the weir has been known and appreciated for a long time. Dr. J. Epper, former Director of the Hydrometric Survey in Switzerland, in correspondence with the writer, in 1914, stated:

"Here we do not use weirs unless they are first calibrated by the current meter. It appears that, no matter how closely the standard weir is copied, differences will creep in due to differences in the distribution of flow above the weir".

In a work published by that Survey, Dr. Epper quotes† the water supply of the City of Berne where weirs gave different results due to varying flow.

For measurements that are not continuous the obvious conclusion from Professor Schoder's experiments is, therefore, to discard the weir and use the current meter instead. If the weir must be supplemented by meter measurements, the use of the meter only is obviously indicated. The weir naturally is more practical in cases where a long series of measurements is to be made. A weir arranged for continuous observation is often found, therefore, which is calibrated beforehand by other means, such as the moving screen. At the

* "Versuche über den Abfluss des Wassers bei vollkommenen Ueberfallen," *Zeitschrift* des Vereines Deutscher Ingenieure, 1890.

† "Die Entwickelung der Hydrometrie in der Schweiz," Berne, 1907.

hydro-electric plant at Ackersand, Switzerland, and at the testing flume of the J. M. Voith Company, in Heidenheim, Germany,* such a combination of weir and moving screen may be found.

As a rule, however, careful observation of standard weir specifications, especially if the velocity of approach is low or, in other words, if the weir is high, will result in good measurements that are reliable. In Table 60 the writer gives the results of hitherto unpublished tests made by William G. Fargo, M. Am. Soc. C. E., and a comparison of relative turbine discharges, as measured by current meters and Francis weir, at the Loud Development on the Au Sable River, in Michigan, run on October 16 and 17, 1913. They are presented by courtesy of William W. Tefft, M. Am. Soc. C. E., Vice-President of the Consumers Power Company, present owners of the dam. The weir was of the standard Francis type, probably the largest ever built. It was 117.583 ft. long; the average weir crest elevation was 216.040 ft. The concrete bottom was at Elevation 207.47, the height of the weir, therefore, being about 8.5 ft. It was solidly constructed of a steel frame set in a concrete foundation and covered with wooden gates having a steel-plate face. The weir crest was made in strict accordance with Francis specifications. The leakage was determined by lowering the tail-race with the penstock gates closed, allowing the leakage through the gates to fill up the penstocks. It amounted to not more than 5 cu. ft. per sec. The head was measured in six places, 21 ft. 9 in. to 23 ft. 6 in. apart, by means of pipe connections placed in concrete pedestals on the bottom of the tail-race. These communicated with six barrels at which the head was measured by six hook-gauges. An aeration pipe was placed on the down-stream side. The flow was measured simultaneously by current meters just up stream from the penstock head-gates.

Two sections were selected, one being measured by Price meters, the other with an Ott meter. The meters had been carefully calibrated in the Naval Tank at the University of Michigan by Professor Herbert C. Sadler. Since the tests of S. Hajós, of the Hungarian Hydrographic Service, published in 1901,† there can be little doubt that a still-water rating is fully equivalent to one in ordinary flowing water, without cross-currents or eddies. Although these tests were made many years ago, they were not widely published and hence the same question has been re-opened many times.‡ The theory and behavior under conditions of oblique flow; the influence of different mountings; and the size of the measuring section of the Ott meters used, had been extensively investigated, and the writer, therefore, could recommend, with full confidence, a comparison test of the Ott meter with the weir.§ The rating

* "Vergleichsversuche mit Flügel und Schirmapparat zur Bestimmung von Wassermengen," O. Lutschg, Berne, 1913; "Die Turbinenversuchsanstalten der Firma," J. M. Voith, *Zeitschrift* des Vereines Deutscher Ingenieure, 1909.

† "Beiträge zur Frage über die Umlaufswerte Woltmannscher Flügel," S. Hajos, Budapest, 1901.

‡ "Characteristics of Cup and Screw Current Meters," by B. F. Groat, M. Am. Soc. C. E. *Transactions*, Am. Soc. C. E., Vol. LXXVI. (1913), p. 819.

§ "Hydrometrische Versuche," J. W. Sandstrom, Stockholm, 1912; "Expériences et Theories sur le moulinet de Woltmann," A. Rateau, *Annales des Mines*, 1898; "Theorie und Konstantenbestimmung des hydrometrischen Flügels," A. Ott, Berlin, 1925; "Die königliche Versuchsanstalt für Wasser und Schiffbau in Berlin," Eger, Dix und Seifert, 1908; "Der normal und schiefgestellte hydrometrische Flügel," Vienna, 1903; "Einfluss der Wandungen auf die Umlaufwerte hydrometrischer Flügel," Vienna, 1899; "Versuche über die Genauigkeit der Woltmannschen Flügel bei schräger Strömung," Frese, *Zeitschrift* des Vereines Deutscher Ingenieure, 1886; and "Untersuchungen über die Umlaufbewegung hydrometrischer Flugel," M. Schmidt, Münich, 1903.

TABLE 60.—COMPARISON OF RELATIVE TURBINE DISCHARGES.

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Revolutions per Minute:			Gate:		Time, Inclusive.			Temperature, in Degrees Fahrenheit.		Remarks.	
			Maximum.	Minimum.	Average.	Inches open.	Opening, in percentage.	Date (1918).	Begin.	End.	Atmosphere.	Water.		
						Full gate, 5¼ in.								
			(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
2 100	2	1	122.6	117.3	120.4	3 25/32	75	Oct. 16	8:52	3:58	51	51	Northeast wind, cloudy.	North wind, rain.
2 138	2	2	121.4	119.1	120.5	3 25/32	75	Oct. 16	4:17	5:28	47	51	North wind.	Rain.
1 364	2	3	121.4	118.8	120.3	2 9/16	55	Oct 17	7:07	11:39	50	51	Northeast wind.	Rain.

NOTES.—The switch-board instruments give results 4½ to 6½% too low.
In Column (43) using h = head in Column (33) and adding leakage per Column (41).
" " (54) an observation at each point took 50 sec.
" " (58) width × depth − ½ × 12 sq. in. = area = 26 × depth − ½ × 12 sq. ft.
" " (63) an observation required approximately 8 periods of 25 rev. each.
" " (67) (width + 6 × ½ in. − 2 × 6½ in.) depth = (26 + 6 × ½ in. − 2 × 6½ in.) depth.
Total observing time for one Epper-Ott meter = 14 455 sec.
" " " " five Price meters = 15 000 ".
Run No. 2 was an experiment on aeration on Francis weir.
Slide-rule computations generally.

NOTES ON WEIR.—Weir crest = Elevation 216.04004 = average of 27 readings { Max. = 216.041. Min. = 216.398.
Weir length = 117.583.
Average bottom at 28 ft. up stream = Elevation 205.28, at weir = Elevation 207.47.
Gamma = 62.3.

TABLE 60.—(*Continued.*)

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Switchboard Instruments.				Tail-water on wheels.	Variation in tail-water on wheels, Column (17) —Column (36).	Head Water.				
			Alternator.		Exciter.				Loss.		At gauging station.		
			Amperes.	Volts.	Amperes.	Volts.			Pond No. 27.	Pond to gauging station.	Average.	Maximum.	Minimum.
			(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
2 100	2	1	488.5	2 575	176.5	86.4	218.12	—0.04	242.76	0.16	242.60	242.62	242.58
2 138	2	2	468.8	2 744	194.5	93.8	218.15	—0.03	242.77	0.16	242.61	242.62	242.60
1 364	2	3	374.7	2 544	141.1	70.0	217.71	—0.00	243.01	0.08	242.98	242.94	242.92

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Head, Q_5.		Vacuum Head on Weir, in Inches.		Tail-water on weir.	Head on Francis Weir.					Head-water on weir.
			H, on turbines.	$\frac{Q_5}{4\sqrt{H}}$ per runner.	No. 35, left.	No. 33, right.		Hook No. 1, right.	Indicating weir gauge No. 2, ⅛ point.	Hook No. 3, No. 4, No. 5, ⅖, ⅜, ⅛ points.	Hook No. 6, left.	Average.	
			(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)
2 100	2	1	24.48	61.7	0	0	213.9	2.119	2.116	2.132	2.115	2.120	218.160
2 138	2	2	24.46	62.6	0	0	213.8	2.135	2.130	2.136	2.150	2.138	218.178
1 364	2	3	25.22	42.6	0	0	213.7	1.674	1.666	1.672	1.684	1.674	217.714

TABLE 60.—(*Continued.*)

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Francis Weir Discharge per Foot.					Total weir discharge, $L = 117.583$ Q_1.	Leakage, Q_3, through weir.	Effective water through unit under test = Q_5.
			Q_a.	Area of channel of approach.	Velocity of approach.	Percentage of increase in Q_a.	Final Q_a'.			
			(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)
2 100	2	1	10.279	12.88	0.8	0.7	10.351	1 217	5	1 222
2 138	2	2	10.410	12.90	0.8	0.7	10.483	1 233	5	1 238
1 364	2	3	7.212	12.43	0.6	0.5	7.248	852	5	857

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Comparison of Francis Weir Discharges by Various Formulas.								
			In Cubic Feet per Second.					As a Percentage of Francis.			
			Francis.	Braschman.	Bazin.	Frese.	Average.	Francis.	Braschman.	Bazin.	Frese.
			(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)
2 100	2	1	1 222	1 235	1 224	1 233	1 228	100	101.1	100.2	101.0
2 138	2	2	.....	.....	.....	.....	.....	....	.....	.....	.....
1 364	2	3	857	869	859	859	861	100	101.4	100.2	101.2

TABLE 60.—(*Continued*)

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Five Small Price Meters.						
			Treatment at Gauging Section.			Area of Gauging Section, in Square Miles.			
			Number of different verticals.	Number of different points.	Number of vertical curves.	With stop-log grooves.	Without stop-log grooves.	Without stop-log grooves and meter rack.	Final net area used.
			(52)	(53)	(54)	(55)	(56)	(57)	(58)
2 100	2	1	18	785	50	610	587	575	581
2 138	2	2	....	....	....	....	....	....	....
1 364	2	3	18	725	45	618	595	583	589

Kilowatt output as per switchboard instrumeuts.	Unit No.	Run No.	Epper-Ott Meter.								
			Velocity.	Flow.	Treatment at Gauging Section.			Area of Gauging Section, in Square Miles.			
			Average as per curves.	Final as per Columns (46) and (47).	Number of different verticals.	Number of different points.	Number of vertical curves.	With stop-log grooves and beams.	Without stop-log grooves.	Without stop-log grooves and beams.	Final net area used.
			(59)	(60)	(61)	(62)	(63)	(64)	(65)	(66)	(67)
2 100	2	1	2.38	1 382	12	100	12	517	498	477	482
2 138	2	2	....	.....	....	....	....	....	....	....	....
1 364	2	3	1.63	960	9	12	9	526	506	485	490

TABLE 60.—(*Continued.*)

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Final Comparison.					
			Velocity.	Flow.	Current Meters.		Meters and Weir.	
			Average as per curve.	Final as per Columns (69) and (70).	Price meter as a percentage of Epper-Ott meter.	Epper-Ott meter as a percentage of Price meter.	Percentage of Francis discharge.	
							Price.	Epper-Ott.
			(68)	(69)	(70)	(71)	(72)	(73)
2 100	2	1	2.54	1 227	112.6	88.7	113.1	100.4
2 138	2	2	.....	.....	.....	.....	.....	.....
1 364	2	3	1.76	864	111.1	90.0	112.0	100.8

curves for the Ott meter had been computed by Dr. Epper of Berne, Switzerland, who wrote that the Naval Tank, at Ann Arbor, apparently was very accurate, as shown by the small probable error. The results of the test which was conducted with great care, are shown in Table 60. It appears that the Price meters gave much too large discharges, but the agreement between weir and Ott meter was very satisfactory. The present Ott meter, Type V, is greatly improved over the meter used in these tests, and gives a rating curve that is practically a straight line from the start.

After it had thus been proved conclusively that the Ott meter is fully equivalent to the weir, no more testing weirs were built, and the measurements were conducted with Ott meters exclusively. In order to make such measurements quickly and accurately, the writer introduced the use of the Tchebycheff formula, as mentioned by Dr. Staus in his recent work on the accuracy of current-meter measurements.*

* "Ueber den Genauigkeitsgrad von Flügelmessungen," A. Staus, Berlin, 1926.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

BASIC INFORMATION NEEDED FOR A REGIONAL PLAN

Discussion*

By Messrs. Donald M. Baker and Harry J. March.

Donald M. Baker,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author has classified the various types of information necessary for the development of a regional plan as: Physical, economic, social, legal, and financial. He has also outlined an excellent program of securing this information. Emphasis has been placed, however, on the physical data necessary and detailed descriptions are given of the methods of collecting and filing it.

Engineers who are connected with planning work are often apt to place too great emphasis on the physical aspects of planning and not enough on other aspects, particularly the legal and financial. Too many plans and planning reports consist, almost in their entirety, of a physical plan with no discussion of the legal and financial methods of attaining the plan.

Use of property within the planned area can be governed by proper zoning, proper streets and traffic arteries through undeveloped territory may be secured by sub-division control, and housing can be regulated, but the necessary public agencies must be set up with adequate powers to carry out this administrative work. Improvements, such as new or wider streets and parkways, park areas, rapid transit and other transportation lines, can be made, but proper proceedings must be initiated for these improvements, rights of way and easements acquired, money must be raised for the payment of costs, and the costs must be prorated on an equitable basis.

Much regional planning work is now done in an advisory way through associations supported by private contribution or the assistance of municipal bodies, but ultimately planning work must be done in each locality by a body with legal authority to enforce adherence to the adopted plan. All States have more or less different laws granting authority to administrative agencies relative to planning work, and Court decisions in each State differ, as do local conditions. Any physical plan outlined for a locality must take these into account, and must also include suggestions for new statutes or necessary changes in existing statutes in order that the proper planning body may be created with adequate authority. Courts are rapidly awakening to the urgent

* Discussion on the paper by Harold M. Lewis, M. Am. Soc. C. E., continued from January, 1928, *Proceedings*.

† Cons. Engr.; Member, Board of City Planning Commrs., Los Angeles, Calif.

‡ Received by the Secretary, December 19, 1927.

need of planning and are adopting a sympathetic attitude toward the exercise of the police power in connection with it. State legislatures as well as city and county governing bodies are realizing more and more the imperative necessity of proper planning and of agencies to administer the work, and it is now much less difficult to obtain satisfactory legislation and sympathetic Court decisions than it was a decade ago.

Planning includes not only the laying out of developed areas and the providing of facilities for future population, but also the correction of existing mistakes and conditions resulting from uncontrolled settlement and development. The latter phase often results in the necessity of acquiring property, rights of way or easements through condemnation, as well as the need of construction work.

Laws in many States, by means of which public improvements must be authorized, are relics of the "horse-and-buggy" days, when the several years required to complete a right-of-way condemnation proceeding was of no moment, and when the costs of improvements and their allocation were simple and not burdensome. The type of improvement was left to the judgment of a few property owners who paid the bill, and who were not in a position to determine whether such type fitted in with a well co-ordinated plan or with the use to which the improvement was to be put. It is just as important that improvements, once they are decided upon as necessary, should be expeditiously carried out and not wait years for Court awards in condemnation as it is that they should be of a type that fits in with a proper plan or that they satisfy the use to be made of them.

Any plan, city or regional, should include a thorough study of improvement laws and should suggest amendments or new laws which will satisfy present-day conditions, and this work should be done at the same time that other investigations are being made and physical data collected, because it sometimes takes a few years to obtain new legislation and have it tested in the Courts.

The entire theory behind the American system of taxation is that payment is made in proportion to the benefits received, and in paying for public improvements resulting from planning, the benefits may be divided into two classes: Those applying to the property directly affected by the improvement, thereby tending to enhance its value from an income standpoint, and personal benefits reflecting on residents who might use the improvement, but whose property is so far removed from it as not to have its value appreciably affected. In short, benefits are specific and general, with no sharp line of division.

It is sometimes hard to classify, in a city or regional plan, those improvements that are "nice" and those that are "necessary". There is, however, always such a thing as inability to pay for an improvement. When the cost of improvements becomes an excessive burden on property owners, it may result in evils to them and to the community greater than the existing ones that made the improvement necessary.

In the "horse-and-buggy" days it was a simple matter to assess the benefits resulting from any public improvement. In general, they were either borne by the entire community, or by property immediately contiguous to the im-

provement. Nowadays, principally on account of the advent of the automobile and other improved forms of transportation, much study and investigation is necessary to assess equitably the costs and benefits resulting from any improvement.

The creation of a park in a neighborhood enhances, very greatly, the value of the immediately surrounding property for residences or apartment houses, but the man living 5 miles away and possessing an automobile is only 15 min. ride from it and probably uses it in a physical sense as much as the man living across the street. The opening or widening of a major traffic artery may be a positive detriment to property owners living upon it, from the standpoint of noise and dirt, and still be a benefit to them and to owners over a large area due to the convenience it affords them in reaching places of work, recreation, or pleasure.

Any improvement, such as a direct traffic artery or a rapid transit line, which brings a place of residence from 15 to 20 min. nearer a place of work, may add several hundred or even several thousand dollars to the value of a single piece of property, and all property so benefited should contribute toward the cost of the improvement. However, the method of payment should be adjusted so that it can be handled by the property owner and still allow him to reap some substantial benefit after he has paid the bill.

This discussion is given not in criticism of the very excellent outline of physical data to be obtained in formulating a plan, but in an effort to emphasize the fact that there are other types of data and other lines of research just as necessary in preparing a regional plan. Since planning work seems to be offering a considerable field to the Engineering Profession it is important to call attention to the importance of other than the physical aspects. Every engineer who has had any contact with public improvement work is more or less familiar with the legal and financial aspects of it, and it is usually far easier for him to understand them than it would be for a lawyer or financier to comprehend the physical and engineering aspect of planning, and therein lies the engineer's opportunity of correlating the three.

HARRY J. MARCH,* M. AM. SOC. C. E. (by letter).†—The author deserves great commendation for the comparatively exhaustive and highly clarified presentation of a seriously complicated subject.

Regional planning has to do with the physical harmonizing and co-ordinating of all the various utilities and resources in the affected area. At the same time, because of its very nature, it causes naturally more or less conflict of persons, ideas, and plans. This is particularly so where glory or money is being dispensed. The various major and minor civil divisions composing the region have their respective officials who are more or less jealous of their prerogatives, not to speak of duties. The regional plan must not unduly trespass upon the respective plans of the units, but must correlate and coordinate them. Therefore, basic data must be obtained and impressively presented, so that it will be compelling in interest and objective.

While in this modern age it is recognized that communities are dependent, one upon the other, even if some of them are remotely separated, the insatiable

* Chairman, Executive, and Engr., City Planning Committee; Chairman, Board of Appeals, Buffalo, N. Y.

† Received by the Secretary, December 31, 1927.

craving of humanity for something different, whether it be in season or out of season and whether its immediate environment be favorable or unfavorable, has caused illimitable unrest and untold strife. The regional planner should first ascertain the existing and potential resources of the several communities within its scope, irrespective of the resultant glory of one community as compared with another, or the profits and benefits accruing therefrom. There must be community team work for effective success.

The life of the several communities must be systematically controlled. Any resources found lacking for the reasonable life of any community should be supplied, if not too prohibitive in cost. In other words, the life of the community should be as well-rounded as that of the individual. There should be varied housing, varied business, varied industry, varied educational and cultural pursuits and varied recreations and diversions.

Large intervening open spaces, which afford the breath of life, as it were, to the adjoining communities, might well form the division area between communities whose specific interests are different, but whose general interests are common.

Planners should know thoroughly the products which may best be developed on the lands within the region, as well as the potential resources of the waters in terms of commercial transportation, power, or recreational phase. For instance, in the portion of the Niagara Frontier Region north of the Lewistown Escarpment, there exists a fertile soil readily adaptable for the larger fruits, peaches, apples, etc.; the middle area, or that to the south of this area, has soil conditions that are favorable for sheep farming and dairying; the southern area is valuable for the smaller fruits, grapes, etc. These several areas partly encircle the large ones of varied industries at Niagara Falls, the Tonawandas, and Buffalo, which are respectively supplied by intervening housing and business areas. The writer desires to stress at this point the great advantage of resources available for the development of the communities, because, with the flagging of one industry, there may be an increase or acceleration of another that is entirely different.

All these conditions constitute the make-up of the self-contained community and produce an enviable economical result that cannot be surpassed by a region lacking varied resources and facilities. The regional plan must be so comprehensive in its scope and compelling in its benefits that the individualized communities will subordinate their selfish interests to those of the comprehensive and economic interests of the whole region.

In the development of regional planning, the basic data to be procured should be that incident to landscape engineering, supplemented by landscape architectural considerations.

In the procurement of data, there perhaps is no better medium than that of a regional planning association for publicity purposes and that of a regional planning board for the technical work involved. The Niagara Frontier Planning Association and the Niagara Frontier Planning Board are typical examples of the effectiveness of such bodies. These organizations have developed an inter-community spirit worthy of emulation, and their members are looking forward to the attainment of some big objectives which perhaps could be obtained in no other way.

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PAPERS AND DISCUSSIONS

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CULTURAL OPPORTUNITIES IN REGIONAL PLANNING

Discussion*

By Messrs. Charles W. Eliot, 2d, Winters Haydock, Gilmore D. Clarke, and Harry J. March.

Charles W. Eliot, 2d,† Esq.—There is a great need for agreed definitions among planners and generally among the public as to what is a park or parkway and as to the proper functions of a park. Confusion concerning these terms has apparently upset Mr. Crawford. A parkway is not a boulevard; neither is it a traffic thoroughfare. The use of the Bronx Parkway in New York by large numbers of automobiles, traveling at high speed, does not make the parkway into a traffic thoroughfare. Perhaps Mr. Crawford only means that more thoroughfares should be wider and attractive. That would be a great advantage, but it would not make them parkways. Is it not possible for the Society, or a similar organization, to put before the public in a forcible way, the differentiation between the purposes of different kinds of parks, and particularly the difference between a park and a semi-public, or other kind of open space?

Winters Haydock,‡ M. Am. Soc. C. E.—An interesting relationship may be discovered between the subject of Mr. Crawford's paper and a recent announcement in the public press, an announcement which undoubtedly is of great significance on account of its economic, sociological, and cultural implications.

The head of a large industry employing an enormous number of workers announced recently that he intends to adopt a 5-day week. That announcement immediately raised a storm of protest on the part of industrial leaders throughout the country, a protest which was the greater on account of the prestige which the author of the announcement, Mr. Henry Ford, enjoys as a successful pioneer in new fields. The plan was immediately denounced as being economically unsound.

* Discussion on the paper by Andrew Wright Crawford, Esq., continued from November, 1927, *Proceedings*.

† City Planner, National Capital Park and Planning Comm., Washington, D. C.

‡ Directing Engr., Dept. of City Transit, Pittsburgh, Pa.

For many reasons it is very improbable that any general change in the working week is an immediate prospect. However, would it not be very rash of any one to declare to-day that it is improbable that the 5-day week will be in the accepted order of things a quarter of a century hence? Suppose that it does become the order of the day? What will that mean? A book entitled, "The Problem of Leisure", has recently been published. That problem is then immediately thrust on the city planner in a much graver aspect. Does not his responsibility then become as great as that of the leaders in education and in other cultural fields whose well-recognized responsibility it is to determine and make provision for the cultural requirements of the people? If this is true, is not one of the most important functions of the city planner and the regional planner the insuring for the future of facilities for the advantageous utilization of this growing leisure of the masses?

GILMORE D. CLARKE,* ESQ. (by letter).†—This paper justifies the wide circulation it has received. Not only is it valuable for engineers, but for the fine arts as well. To-day, these professions have been brought face to face with tremendous problems in connection with the planning of regions, cities, and counties, and of all their component parts.

The problems cannot be solved by members of any one profession. The multiplicity of detail which enters into regional planning are such that no single profession can ever master it. Mr. Crawford brings the Engineering Profession to task for its accomplishments in connection with city planning in the Nineteenth Century. He states‡ that,

"They [engineers] have only themselves to blame, that the public, weary at last of the drab, stupid, stale rectangularity of the checker-board cities, which were all the civil engineers of the century produced, relegated them to the subordinate position of drawing the working plans conceived by others".

This is true, but there are many engineers who now realize the necessity for collaboration with the professions of the arts, in order to effect results in the planning of areas and in the design of structures which will be more satisfying to others as well as to themselves.

A concrete example of collaborative work on the part of members of the professions of engineering, landscape architecture, and architecture, is the following: A Commission was organized in 1922 to prepare a park and parkway plan for Westchester County, New York, which lies just north of New York City between the Hudson River and Long Island Sound. The area of the county is about 500 sq. miles, the richest suburban area in the country. The working organization is made up of a Chief Executive Officer, whose title is "Chief Engineer", a Deputy Chief Engineer, whose duties have to do with engineering design and construction, and a Landscape Architect charged with the planning and design of park and parkway areas and the design of park structures. Architects are called upon to design all buildings which do not come under the category of "minor structures". In addition, there are

* Landscape Archt., Westchester County Park Comm., Bronxville, N. Y.
† Received by the Secretary, December 12, 1927.
‡ *Proceedings*, Am. Soc. C. E. September, 1927, Papers and Discussions, p. 1529.

other departments, such as legal, land purchase, maintenance, and secretarial and accounting departments.

There has been no disposition on the part of the efficient engineers in this organization to complain of the injection of the landscape architect and the architect into this regional planning problem. As a matter of fact, they have welcomed these representatives from the arts, and the results of this co-operation are noteworthy.

In the matter of bridge design, the writer has had an interesting experience in working out more difficult problems with the Commission's structural engineer. There has never been any disposition on the part of this engineer to accomplish work in design that normally is in the field of architecture. The writer believes that, because the bridges designed for the Westchester County Park Commission are the result of collaborative effort, they represent the best there is in modern bridge design in the world.

Mr. Crawford holds out some hope that the engineer may some day become competent to design artistically—to be a "dreamer". The writer doubts it. The mind of the average engineer is trained to precision, to the definite practical working out of problems, to things concrete. Such a mind does not easily re-act to the artist's business of dreaming and imagining. What must be accomplished is to bring these minds together, for one individual to realize the shortcomings of the other, to the end that, when working together on a single problem, it will represent joint efforts and will produce the finest results.

Engineers are practical men, and it is hoped, for the sake of engineering, that they may remain so. Dreaming must be left to other souls; and then the dreamer and the practical man, by getting together and collaborating, will accomplish more than if the attributes of both were centered in one individual.

HARRY J. MARCH,* M. AM. SOC. C. E. (by letter).†—Mr. Crawford has evidently not yet sensed the relative importance, first, of economic design, and, second, of pictorial design; or, in other words, first the work of the engineer and then that of the architect. There must be first a substantial structure that will serve the highest utility purposes and then let it be painted. The writer is not unmindful of the character of utility in the artistic design, but it is not an absolute necessity. To illustrate, he has in mind a landscape architect's dream of a development which, while beautiful in its effect, utterly disregarded economic traffic conditions. The architect had to be shown by the engineer the relative value of the traffic and the artistic phase, and he finally subordinated the latter to the former. There is a need to stress the importance of co-ordinating the work of the architect and the engineer. The architect should be more of an engineer and the engineer should be more of an architect. Co-ordination is the keynote of the whole development of regional planning as well as of municipal planning. Would that "the powers that be", controlling as they do the extent and intent of public improvements, could realize the value of such development!

* Chairman, Executive, and Engr., City Planning Committee; Chairman, Board of Appeals, Buffalo, N. Y.

† Received by the Secretary, December 31, 1927.

This phase of the inter-relationship of the architect and engineer is an important one in the development of regional planning, particularly where bridges are to be erected and county parks or forest preserves established. Every one is quite familiar with the terms, "City Engineer", "City Architect", and "County Engineer", but one seldom hears of a County Architect. The regional area in its development should bear the impress of the architect to the same proportional extent as the municipal area bears the impress of the City Architect. The idea could be best comprehended, perhaps, under the term, "landscape engineering", rather than "landscape architecture".

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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HISTORIC REVIEW OF THE DEVELOPMENT OF SANITARY ENGINEERING IN THE UNITED STATES DURING THE PAST ONE HUNDRED AND FIFTY YEARS

A SYMPOSIUM

Discussion*

BY MESSRS. GEORGE H. FENKELL, N. T. VEATCH, JR., E. R. JONES, AND LEONARD GREENBURG.

GEORGE H. FENKELL,† M. AM. SOC. C. E. (by letter).‡—Mr. Fuller has made a very complete and interesting résumé of the advance in water supply practice during practically the full life of water supply systems in the United States. To the student, historian, and others, who may wish to pursue the subject in further detail this paper will serve as an authoritative reference and starting point.

The author's quotation§ from a paper by the late J. James R. Croes, Past-President, Am. Soc. C. E., regarding the development of a method of pumping water direct to a distribution system and of controlling pressures at the point of pumping, is very interesting, as this method is still in common use in many cities and towns, particularly in the Middle West. The development of the motor-driven fire-pump has eliminated, to a great extent, the necessity for excess ranges of pressure on the distribution system, which pressures were apt to have a bad effect on the plumbing systems of the buildings served with water. The increasing use of elevated storage tanks and the gradually increased capacity of those being built also tend to smooth out some of the

* Discussion of the Symposium on Historic Review of the Development of Sanitary Engineering in the United States During the Past One Hundred and Fifty Years, continued from January, 1928, *Proceedings*.

† Supt. and Gen. Mgr., Dept. of Water Supply, Detroit, Mich.

‡ Received by the Secretary, December 21, 1927.

§ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1587.

pressure variations which always accompany the use of this system of distribution.

For the purpose of completing the record it is to be regretted that Mr. Fuller did not extend the data on maximum steam pressures used for pumping water beyond the year 1906, and, at the same time, add information relative to pump efficiency and duty. For completeness, some recent performance of steam turbine and electric motor-driven centrifugal pumps would have been interesting, as these are fast coming into more general use.

The development of intakes for lakes and rivers having small variations in level has probably been more marked on the Great Lakes and in tributary waters than elsewhere. This has been due to the great extent of this region and the large population making use of these waters. Throughout this district, winter weather is generally severe, and serious trouble from ice may be expected even in the southernmost part. Frazil, or needle ice, and the grounding or piling up of surface ice are usually most troublesome, and dangers arising from the heavy lake traffic must also be considered. These factors materially affect the design and maintenance of intakes on the Great Lakes.

The simplest intakes consist only of a pipe line laid on the lake bottom, and may terminate in an upturned elbow, or tee, covered with a screen or straining device of some kind. The elbow may have no support, or rip-rap may be piled around it. In some cases the pipe terminates in two or three elbows. A more elaborate design of the inlet end of the pipe (and this type was used by the City of Detroit, Mich., in 1858) consists of a boiler-plate cylinder having a dome top perforated with a great number of ½-in. holes. Another variation has a cast-iron or steel hood or cage placed over the inlet elbow or toe of the pipe line, which is pierced with holes ½ in. or more in diameter. Designs which included no protecting crib were probably in more extensive use formerly than now.

At Toronto, Ont., Canada, the inlet is carried above a protecting crib and, for the older intake, is protected by a cone consisting essentially of twenty-eight 1¼-in. round bars, spaced equally around the circumference, and for the new intake by a steel plate supported 30 in. above the lip by 1¼-in. round rods, spaced 8⅛ in. on centers.

For the larger intake pipes it is customary to protect the inlet end by a submerged crib. There is considerable variation in the construction of these cribs. The larger ones have usually taken the form of a square or octagonal timber crib having a solid floor. They are divided into compartments by the cross-timbering. The conduit is carried through the cribbing above or below the floor to the central compartment or well, terminating in an elbow. In some cases, to provide for future extension, the conduit is carried across the entire crib, the end being closed with a blank flange outside the crib and a tee provided at the central well. The crib is towed out to its final position and sunk, and as many compartments as are necessary to hold it down are filled with broken stone or concrete, or both. The usual practice is to cover the central well, and often the four compartments adjoin-

ing the sides of the well, with a grating of plank placed on edge and spaced about 2 in. in the clear. In some cases, as at Toronto, the pipe is carried above the crib and the inlet protected.

The submerged type of intake structure is usually built in connection with a pipe conduit, but at Gary, Ind., Crib No. 5 at Cleveland, Ohio, and the new intake at Milwaukee, Wis., the conduit is a tunnel. Practically all exposed intake structures have been built in connection with tunnels, but that at Montreal appears to be the exception. These structures are usually limited to depths of 40 ft., or less. Crib No. 3 at Cleveland is in the deepest water, about 49 ft.

An exposed crib consists essentially of a central well enclosed by massive walls of sufficient thickness to give strength and stability to resist the forces of wind, wave, and ice action. The intake shaft of the tunnel draws the water from this well, and ports in the crib walls admit the water to the well. These cribs are built on shore, launched, towed to position, and sunk, the bottom having been leveled off by dredging.

There has been considerable change in the design of exposed intake cribs. The City of Chicago, Ill., with its large population and heavy shore pollution requiring the location of its intakes at increasing distances from shore, has taken the lead in developing exposed intake structures. Two-Mile Crib, completed in 1867, was the first. It was built entirely of timber, pentagonal in shape, with 58-ft. sides, and a height of 40 ft. The central well was of similar shape, and the seven compartments of the wall space were filled with rubble during sinking. The Hyde Park or 68th Street Crib is of similar construction, but hexagonal in plan. These two intakes are peculiar in that they are enclosed by a heavy breakwater. Crib No. 4, the first exposed crib at Cleveland, built prior to 1890, was pentagonal and was constructed entirely of timber. The Four-Mile Crib, built during 1890-94, the Lake View Crib, completed in 1896, and the Carter M. Harrison Crib, built about the same time, at Chicago, mark a change in construction. They are all circular or polygonic in plan and have a base of heavy timber grillage through which the ports pass. On this base are two concentric steel shells with radial bulkheads forming compartments which, in the case of the Four-Mile Crib, were filled with concrete. Crib No. 3, at Cleveland, built about 1898, is very similar in construction.

The Detroit intake crib, built during 1905-06, was constructed entirely of timber and the compartments were filled with concrete. Its shape is an elongated octagon. The City of Buffalo, N. Y., completed a crib in 1910 having two concentric steel shells with no bottom. It was divided radially into compartments, a number of which were closed at the bottom with timber for buoyancy when towing the crib to position. The crib was landed on a concrete foundation ring resting on rock, and then filled with concrete.

The Wilson Avenue Crib, at Chicago, completed about 1918, presents another variant. There were two concentric steel shells with radial compartments, but no floor. The bottom of each shell had a cutting edge. For

buoyancy, the ports were bulkheaded, and buoyancy tanks were built between the shells. When sunk, the steel shoes entered the clay and sealed the bottom. Concrete was used for filling.

Exposed cribs are usually provided with a more or less elaborate superstructure with quarters for attendants. Except for the Wilson Avenue Intake, Chicago, the Linwood Avenue Intake, Milwaukee, and the Buffalo Intake, the tunnels have been driven through clay.

Ice is a source of much anxiety and trouble for many who maintain intakes on the Great Lakes, because it may greatly reduce, and, at times, cut off the flow. Sheet ice and frazil ice are usually the source of trouble, sheet ice chiefly after breaking up, for it is then carried by wind and current and may form ice jams obstructing the flow to the conduit. Frazil ice, by adhering to the sides of inlet openings and ports, gradually builds up and becomes a serious obstruction, often cutting off the flow entirely.

Various remedial measures have been used to overcome ice troubles, particularly from frazil ice, but none has been uniformly successful. Experience has shown that the depth of the port opening below the surface has a marked effect. The depth depends on local conditions, but 35 to 40 ft. will usually give freedom from trouble with frazil ice. The velocity of flow through the ports has also been reduced on various intakes in an effort to minimize the tendency of frazil ice being drawn into the ports. Velocities of less than 0.5 ft. per sec. have been used, but there is no available data as to the maximum velocity which will not overcome the buoyancy of this ice.

Back pressure on the intake ports of submerged cribs has been used more or less effectively to remove adhering frazil ice. Steam and compressed air have also been used, and dependence, in some cases, is placed on sufficient storage to tide over the demand until natural causes assist in removing the obstructing ice. On exposed intakes, particularly at Chicago, a crew of men is stationed on the crib to combat the ice. Pike poles, steam, and small charges of dynamite are used for this purpose.

In shore wells where ice frequently clogged the screens, traveling screens replacing the former stationary screens, have eliminated trouble in many cases.

The steam-operated power plants on the Great Lakes also use large quantities of water for condensing. The maximum consumption per plant of the Detroit Edison Company ranges from 184 000 to 500 000 gal. per min. The average is more than 2 gal. per min. per kw. The intake arrangements of the power plants are quite different from those of the water supply plants. The water usually comes through a short canal, or a forebay, which may or may not have a boom across the entrance to deflect floating débris and ice. At the Avon Plant of the Cleveland Electrical Illuminating Company, and at a plant in Milwaukee, a lagoon has been built on the lake shore using heavy rip-rap for the enclosing dikes.

Traveling screens are usually installed at power plants to remove débris and ice. These plants are also arranged so that the heated cooling water can be discharged into the forebay and thus prevent trouble from ice.

N. T. VEATCH, JR.,* M. AM. SOC. C. E. (by letter).†—The subject of street cleaning and the disposal of refuse, discussed by Mr. Greeley, is one which, in a way, may be termed an "orphan child" of the Engineering Profession. Few engineers have had the desire, or perhaps the nerve, to make any effort to study the subject from its engineering or scientific angles, and, as a consequence, the profession does not have the wealth of engineering data and background of precedents in this particular field that it enjoys in many others. An engineering writer who is unusually well informed on the subject once offered, in substance, the following statement, as one of the reasons for its present status from an engineering standpoint: "The question of collection and disposal of refuse has been made a 'political football' so to speak, and there is an almost universal lack of realization on the part of city officials that the subject is really an engineering matter".

This assertion is undoubtedly true, as is evidenced by the fact that, generally speaking, this particular division of municipal service is handled by branches of the city government not connected with the engineering department, and with comparatively few exceptions it is given no engineering advice or supervision. As a result, there are numerous examples of failures in practically every method of handling the problem, and practically every city official, in whose department the refuse collection and disposal is handled, feels that he has started his duties as a public official with a veritable "stone hung around his neck".

The lack of proper methods in street cleaning and the collection and disposal of refuse has been, and still is, responsible for the loss of many lives among the urban populations of this and other countries. This is true to a degree sufficient to challenge the interest and effort of every one interested in sanitary science. The fact that, at best, the refuse problem is a difficult and unpleasant one, due to its very nature and its political entanglements, is perhaps one of the causes for its not receiving the attention it should have from the Engineering Profession in general. However, the difficulties that the problem seems to carry with it are causes only, and should not be made into excuses. In spite of difficulties, an immense amount of scientific engineering thought and study has been given to the problem, and there are numerous examples of outstanding service on the part of conscientious city officials and members of the Medical and Engineering Professions. There are also numerous examples of cities having efficient and successful handling of their street cleaning and refuse problems, and it is pleasing to note that most of them have had scientific and technical advice, and that they have followed it.

Much valuable data have been accumulated both in this country and abroad, valuable service has been rendered, and valuable results have been obtained by a rather small coterie of the Engineering Profession. Mr. Greeley is one of that number, and the profession is indebted to him as well as many other contributors. He is undoubtedly correct when he states‡ that, "efficiency of operation and progress in the future will depend in large measure on the character and training of municipal governments."

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* Cons. Engr. (Black and Veatch), Kansas City, Mo.

† Received by the Secretary, December 31, 1927.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1627.

In Kansas City, Mo., the question of garbage collection and disposal was for years a "thorn in the side" of the city officials. The service has been, and still is, obtained through private contract. For a long time, the contractor was paid a lump sum per year, the amount being increased from time to time as the city increased in population. This plan did not give the best results, as the house service was unsatisfactory. The contractor fed the collected garbage to hogs, and in consequence a considerable amount of material that goes into the usual house garbage was rejected by the collectors, and numerous complaints of non-collection were made. Such results, perhaps, could be expected where the contractor's co-operation depended in no way on the amount of garbage collected.

In an attempt to solve the problem, the city officials found themselves confronted, not only with the question of proper collection, but also with one of disposal, because the location of the hog-feeding pens was unsatisfactory. Due, somewhat, to lack of proper engineering guidance, the city at one time tried to enter into contract with a private disposal company on the basis of building, at the city's expense, a plant to dispose of the garbage by a patented process, which to that time had not demonstrated its feasibility. This action was stopped by injunction. Later, the city had engineering specifications prepared, and called for bids including those for both collection and disposal. The contractor was to select his own method in the latter. The successful bidder turned the matter of mechanical garbage disposal over to a sub-contractor and agreed to assume all expense of installation and responsibility for the operation of the plant.

As soon as the contractor had his work under way, and his routes organized in a preliminary manner, the city had an engineering survey made, and a report prepared covering the services included in the contract. This report included a careful study of the city, its population distribution and concentration, character and amounts of garbage from different sections of the city, manner of house treatment, operation and routing of collection units, together with all other features entering into the problem. As a result of this engineering study, and a rather unusual effort in co-operation on the part of the contractor, marked improvement in the collection service very quickly appeared.

The contract calls for payment by the city on the basis of tons of garbage collected, and carries with it a definite penalty clause covering complaints for lack of service. This plan has resulted in continued good service.

The patented process used for garbage disposal is known as the "Pan-American process", mentioned by Mr. Greeley.* While the plant has not been in operation long enough to prove its practicability definitely, the process, if successful, will be a decided benefit, and its feasibility is looked forward to with a great deal of interest.

Although there may be considerable question as to whether the best service can be given under municipal operation or private contract, there is no question but what Kansas City benefited materially from the scientific manner in which its garbage disposal problem was handled.

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1624.

E. R. JONES,* ESQ. (by letter).†—The drainage policy throughout the United States has changed since 1920 from one of extensive to one of intensive drainage. By the former method is meant the practice of constructing large outlet drains for great undeveloped areas for the purpose of creating more farms. Intensive drainage means the drainage, usually by means of drain tile, of the wet spots within or at the edges of existing farms and fields.

Prior to 1920 both types of drainage were making steady progress. The agricultural depression beginning then made extensive drainage unprofitable. The demand for land, particularly new land, became less, the price of land fell, and new reclamation practically ceased.

This was not true of intensive drainage. In the face of high labor costs, and low prices for agricultural products, the farmer had to look around for means of reducing the cost of production in order to keep up his profits. He found that he had to keep two hired men during the cultivating season to give adequate cultivation to a 40-acre cornfield if it was mutilated with wet spots and swales. The fields were too wet so much of the time that he had to have a double crew ready to work fast when these spots did dry up.

Then he got tired of feeding and paying the extra hired man. He began to tile-drain the wet swales in those fields for the chief purpose of reducing the cost of cultivation. Increasing the yield was a mere incident. He found that there were commonly twice as many hours during June when a well-drained cornfield could be cultivated than were available in a field mutilated by wet swales. This meant that one man could cultivate as much corn in a well-drained field as two in a poorly drained one. That is why farmers are draining the wet spots in their fields.

One of those farmers is a resident of Reedsburg, Wis. He can use a tractor on his farm after draining the wet spots in his fields. He has paid for the tractor with the money he saved by discharging the extra hired man and with the increased yield of corn. The drain tile is his remedy for the high cost of farming. Fifty-eight carloads of tile were laid in his township in 1926.

Fortunately, intensive drainage does not cost as much as extensive drainage. With the latter, 200 rods of 5-in. tile, laid in trenches 3 ft. deep, discharging into a ready creek, at a total cost of $2 per rod, will commonly dry up all the wet swales in a 40-acre field. This $400 expenditure reflects a value on the entire field, because the wet swales made the slopes between them inaccessible in stormy weather. On extensive areas of wet land, not only do outlet ditches have to be dredged, but lines of tile 4 rods apart on the whole area may be necessary, at a total cost of $100 per acre instead of $10. That does throw the dredge or big-tile outlets into the discard. They are still necessary, however, in many old communities where a sluggish creek now forms an inadequate outlet for the lines of tile on the wet swales in the cultivated fields. Likewise, the three-cornered fields west of Brodhead, Wis., can be made rectangular and economical only by improving the channel of

* State Drainage Engr., Madison, Wis.

† Received by the Secretary, December 27, 1927.

Juda Creek for 4 miles. This project is now under way. Because of the flashy run-off from the surrounding hills, a big ditch is better in that case than a big tile.

LEONARD GREENBURG,* ESQ. (by letter).†—The steps by which progress has been measured in the field of heating and ventilating during the last one hundred and fifty years are clearly described by Dr. Palmer.‡

It seemed certain in the earlier days that the discomfort one felt when placed in a poorly ventilated room was due to the vitiation of the atmosphere, brought about either by the production of subtle exhaled toxins, or by the accumulation of carbon dioxide. It has since been proved that these theories are incorrect and to-day most people are convinced that the discomfort is due primarily to overheating.

Every one must eat food in order to obtain the energy necessary for bodily activity. This energy is obtained from the oxidation of an amount of foodstuff capable of yielding approximately 1 to 3 000, or more, calories per day. The heat of this oxidation is liberated within the human body and in the body tissues. Some of it is needed to keep the body warm (98.6°), but a large portion must be wasted. This is done by the usual means (conduction, convection, radiation, and evaporation) from the lungs and the surfaces of the body. If the environment is too warm, the body will have a difficult or almost impossible task in dissipating its excess heat, the heat-regulating and vascular systems of the body will be overtaxed, and discomfort will ensue. The problem of ventilation, in the light of present knowledge, resolves itself chiefly into the task of warming rooms without overheating them. To be sure, the problem of air supply is still of importance, for it is by this means that the heat is removed from large spaces, such as auditoria or theatres. To be specific the room must be comfortable when people enter it, and must remain so, in spite of the continuous contribution of heat from the people, as well as that from the heating sources which are in use. It is the regulation of these heating sources and, if necessary, the removal of body heat which must be accomplished if the environment is to be kept at the proper temperature. All this must be accomplished, as Dr. Palmer states,§ without the production of unpleasant drafts of cool air.

The engineer has been accustomed to deal with physical structures and materials which he can measure and weigh. To him the problem of ventilation presents one peculiar aspect, for the test of ventilation in the room is the production of a condition which shall feel comfortable to the majority of the occupants. Feeling is something new to the engineer; it is a yardstick that is more familiar to the physician. Fortunately, it is possible to translate feeling into temperature for it has been found that healthy sedentary people usually feel uncomfortable indoors in the winter when the air temperature is more than 70 degrees.

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† Received by the Secretary, December 23, 1927.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1641.

§ *Loc. cit.*, p. 1645.

In addition to the importance of the subjective sense of comfort as a criterion of ventilation there appears to be one other aspect of the problem to which the ventilating engineer attaches insufficient importance, namely, the necessity for variability in the atmospheric conditions to which the body is exposed. The human body is peculiar, in that conditions which are continuously uniform are monotonous and distasteful to it. For this reason, the window method of ventilation with its intermittent breezes of cool and warm air, is much more pleasant than the method of closing windows and ventilating by means of the fan in the basement.

One of the real stumbling blocks in the progress of ventilation has been the indifference of the ventilating engineer to the advances which have been made in the field of physiological research. In spite of established facts there are still many ventilating engineers who demand an air change of 30 cu. ft. per person per min. It is important to note that if this change is supplied at a higher temperature than 70°, it may serve to overheat the room and produce discomfort, whereas if it is too cold, it will chill the occupants of the room. The temperature of the air is the important consideration which many still overlook.

Ventilation laws prescribe the supply of 30 cu. ft. per person per min. Few of them definitely state the temperature at which the environment should be maintained. It is obvious that they overlook the all important part of the problem. In addition, they actually impose a financial burden on the owners of the buildings and, in return, yield atmospheric conditions which, according to the studies of the New York State Commission on Ventilation, are responsible for many respiratory diseases.

Dr. Palmer's tabulation* of the changes in ventilation likely to take place in the immediate future, is excellent. The writer suggests a slight change in the order of importance of these developments. To a very great degree engineers lack an understanding of what ventilating systems, once installed, will accomplish. In order to design new systems which will work with the utmost efficiency it is necessary to know the working characteristics of those now in use. On a basis of such studies, engineers will be enabled to alter designs and take into account some factors which, perhaps, are now overlooked and, what is still more important, they will learn how to operate existing plants. It will be possible to instruct those in charge of operation just what to do, when atmospheric conditions arrive at the optimum, in order to maintain these conditions. A ventilation plant should not be compared with a bridge or a building designed by an engineer because these structures usually require little or no operation, but it should be compared with the water purification or sewage treatment plant; these require expert operation and receive it as a matter of course.

It is to be hoped that, in the future, the ventilating system will be given more expert supervision and operation. If this is lacking one naturally turns his attention to the use of automatic temperature control apparatus and, without question, the improvements to be expected in this field should do much to

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1647.

increase operating efficiency. At first glance the problem may appear to be a simple one, but more careful study and reflection will prove it to be complicated. It must be borne in mind that direct sources of heat, such as radiators, are large heat reservoirs, and when the steam is cut off they still continue to give off heat to the room atmosphere. This suggests, at once, that radiators should be turned off somewhat below the upper limit of the comfort temperature. The satisfactory operation of thermostatic control devices depends in a large measure on adjustment and maintenance, which again emphasizes the need of trained personnel.

The field of industry presents the most interesting and important tasks for the ventilating engineer. The ventilating problems of industry are manifold, and they continue to grow in number each year. Plants for the removal of heat, for refrigeration by means of cold air, and for the removal of noxious gases, dusts, and poisons, require the services of the ablest workers. Exact knowledge of the necessary elements in designs of this type are in the hands of relatively few, and new data are essential as newer problems arise. Furthermore, these problems are complicated by industrial conditions and plant processes. They are important both from the health and economic standpoint, and should receive increased attention from the Engineering Profession.

Problems of cooling in the warm months of the year and re-circulation during the heating season depend in a large measure for their solution on the economics involved. It appears to the writer that the ventilating engineer can cool the ordinary auditorium very satisfactorily, provided the rather large expense involved is no handicap. Re-circulation can well be practiced if the economic return is justified by the design required. That improvements in this latter type of system are required there is no doubt, but these are largely of the nature of thermostatic temperature control discussed previously. The odor problem, which is of importance in this connection, may be solved partly by the careful control of temperature and humidity (odors are more pronounced at higher temperatures and humidities) and, in some measure, by the admission of outdoor air.

The chemical and not the physical viewpoint of the problem of ventilation may be regarded as the one single important obstacle in the way of ventilation progress. When this is completely eradicated progress in this field will be more rapid than it has been in the past.

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INDUSTRIAL WASTE DISPOSAL
A SYMPOSIUM

Discussion*

BY EDMUND B. BESSELIEVRE, M. AM. SOC. C. E.

EDMUND B. BESSELIEVRE,† M. AM. SOC. C. E. (by letter).‡—Broad-minded and far-sighted ideals have been the background of the Department of Health of the State of Pennsylvania for a long time. The Keystone State, perhaps the first, and by far the foremost, State of the Union to recognize and accept the doctrine of co-operation between State and industry, has progressed far in regard to the allocation of streams to their most economic uses, the recognition of the basic rights of industry, and the discrimination between selfish interests and those necessary to the State's welfare. As have other States, Pennsylvania has some citizens who believe that their pleasure or convenience is paramount to any other consideration. It makes a good argument to claim that fish are killed because of stream pollution. However, is it right to stifle or penalize an industry which produces millions of dollars of revenue and employs thousands of people, for the sake of providing a few fishermen the right to pursue a sporting proclivity? No! The State of Pennsylvania (and here much of the credit is due to W. L. Stevenson, M. Am. Soc. C. E.) has been wise enough to form a Board to have jurisdiction over its intra-state waters. This Board, in turn, has promulgated wise rules that fully weigh the inherent rights of the individual as to industry and health.

Co-operation with industry, in assisting industrial groups to solve problems of pollution peculiar to that industry, has been another feature of Pennsylvania foresightedness which has been followed elsewhere. It is a basic fact, known to those who are closely identified with stream pollution, that industry

* Discussion on the Symposium on Industrial Waste Disposal continued from January, 1928, *Proceedings*.

† San. Engr. and Mgr., Eastern Territory, San. Eng. Div., The Dorr Company, Engrs., New York, N. Y.

‡ Received by the Secretary, December 30, 1927.

is prone to balk at stringent requirements arbitrarily made and enforced. Waste treatment at its best is an expense, and an industrial plant owner cannot be blamed for holding off the fatal day as long as possible. Therefore, an inclination to assist by working with the industry and devising a sensible program of experiment is bound to be received with satisfaction. The only fault the writer can see in this policy is that it may lead to procrastination; that is, an industry, if faced with an expensive treatment, could continue its experimental work indefinitely and thus stall eventual compliance with the law. That is not true in the co-operative measures thus far undertaken, because the larger industrial groups have sensed the inherent justice of discontinuing pollution, and are genuinely ready to find a reasonable way of compliance. Much good can be accomplished by this industrial group work, as the financial burden entailed on any one member is small, and at the same time sufficient funds can be obtained to enable really worth-while work to be carried on. It should be encouraged, provided satisfactory and economical methods of treatment for the waste in question are not already known.

The third, and by no means the least important, co-operative measure developed in Pennsylvania has been the fostering of joint action with adjoining States over interstate waters. Obviously, no relief can be obtained on a stream which divides two States, without concerted action by both States involved. The co-operative arrangements among the States of Pennsylvania, Ohio, West Virginia, and New Jersey, and the work that has been done on the rivers which separate these States, is the best answer to the value of this phase of clear-sighted public health work. It is hoped that nothing will be allowed to interrupt or halt this good work so essential to the welfare of all.

Setting a standard of treatment or degree of purification is one of the best ways to get action, provided the standard is logical and practicable. Oil refinery wastes, as discussed by Mr. Weston,* have not been given the extensive study that other wastes have received, and the efforts have been to extract as much of the oil and tar as possible by gravitational means. This simple treatment has sufficed in many cases, mainly because the volume of the diluting water receiving the wastes has been great. There are cases, however, where greater results must be produced and sufficient work has been done on wastes from large refineries to show that Nature can be assisted at moderate cost.

Precipitation of tars and coagulation and flotation of lighter oils may be accomplished chemically, electrically, and mechanically. With proper sedimentation basins provided with mechanical skimming devices and means for collecting the deposited sludge, this process can be made continuous and will occupy little space. Assumptions, however, are dangerous in industrial waste practice, and tests should be made in each case to determine the best and most economical method to adopt.

Mr. Crichton† is correct in his premise that treating coal mine wastes with lime or other chemicals is simply shifting the burden from the shoulders of the mine operator to the shoulders of the public through its water filtration plant. The question arises as to the relative economy of neutralizing acidity

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1653.
† *Loc. cit.*, p. 1656.

at one place, and reducing the increased hardness at another place, as against carrying out all the treatment at one place. The cost of treatment or neutralization mentioned, 15 to 25 cents per 1 000, is not excessive as compared to the cost of treatment of other wastes; but although a few cents per 1 000 gal. may seem small, when multiplied by thousands of thousands the bills pile up swiftly.

At this point, it is proper to call attention to the fallacy of considering industrial waste treatment problems on the basis of cost of treatment of a small volume tested without ascertaining the true cost of the total volume to be handled in a given case. It does not seem expensive to throw a little chemical, perhaps a few grains, into a beaker of waste to achieve a result; but multiply those few grains by the number of gallons required to be treated, and the small pinch of chemical becomes a carload or a ton of the material. Investigations should be carried out with this clearly in mind, because frequently different methods may be required if one proves to be impracticable from a cost standpoint. Also, at this point, the relative initial cost of the plant, as compared to operating expense from the viewpoint of the chemical bill, must be visualized. If ground area is available, and experiment shows that a larger plant will require less chemical, and the interest charge on the investment for the larger plant is less than the cost of chemical for the smaller one, then the larger plant is the most economical.

Referring to the Melcroft problem,* the writer was consulted on this, and the investigations showed that the wastes could be neutralized at about 16 cents per 1 000 gal., but that the hardness of the water was increased. This fact was reported to the Melcroft Company as was duly proper.

Paper mill wastes, discussed by Mr. Spence,† are one of the few industrial wastes which offer a hope for economical recovery of usable material. To attack industrial waste problems in general from the standpoint of showing a profitable recovery of some ingredient is wrong in principle and is misleading. Very few wastes are entirely free from potentially valuable constituents, but the cost of reclaiming these substances is very often "a delusion and a snare", and new raw material may frequently be purchased at less than the cost of reclamation. Expensive plants have been built to recover waste materials and after operating for short periods at great cost close down. Industrial waste treatment should be attacked from the viewpoint of producing the desired result and of letting any recovered product be used as a partial stand-off of the cost of treatment. Do not spend the savings before they are made, but be happily surprised if a slight return comes from the operation.

A great deal of work has been done in the treatment of paper mill wastes, and it has been demonstrated to the satisfaction of some very large companies that sufficient pulp can be recovered not only to warrant the original expense of the plant, but to amortize the entire cost in a period of from one to three years. Recoveries of 95% and more of the wasted pulp are common, and even in cases where the pulp is valued at 1 cent per lb., it has been found to be a paying investment. In those cases of high-grade papers where pulp

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1663.
† *Loc. cit.*, p. 1667.

is worth 6 to 7 cents per lb., or more, the writing off of the initial investment is a matter of months. Carefully designed and constructed save-alls are in operation in many mills and that the results are satisfactory to the owners is attested by the purchase of duplicate plants for other mills. Sometimes, it is necessary to add alum to produce efficient results, but as in some high-grade papers, alum is used in the normal process of manufacture, this does not destroy the value of the recovered pulp.

In deciding between two competitive devices, one of which produces a much higher recovery by using alum or some other coagulant and the other of which achieves a lesser recovery without the use of alum, there are two factors to weigh: First, if the higher recovery is necessary to satisfy health authorities, then the unit producing the lesser recovery is out of the question, despite its possible lower cost; and, second, if one unit is cheaper in first cost than another, but requires the use of alum to procure a high recovery, which cost must be capitalized and added to the first one, and the other unit does not, the relative recoveries must be considered and evaluated. If a unit does not produce the required result it is bound to be an unsatisfactory investment despite the apparent saving of a few dollars a year. Guaranteed results are what count, and statements of expected results that are not based on test, should be looked at askance.

Relative to the paper by Messrs. Howalt and Cavett,* the treatment of tannery wastes may be properly subdivided into two classes rated according to the difficulty of achievement of the object, namely, (1) partial treatment; and (2) complete treatment. It has been the writer's experience that the second of these objects is the one least often sought for, at least, this is true at the present time. It may be that, owing to the size and the nature of the streams in Pennsylvania and the location of the tanneries, the necessity for complete treatment in that State exceeds any other demand, but elsewhere this is not so.

Tannery wastes are relatively easy to treat from a standpoint of eliminating or reducing the elements of nuisance—solids, hair, sludge banks, scum, and odor. At present, a large number of installations in various parts of the country are satisfactorily achieving the purpose for which the plants were installed.

It is a recognized fact that a great deal of the hair, fleshings, and small leather scraps, may be most economically and satisfactorily removed by mechanical fine screens. This feature has been adopted at some of the largest and leading tanneries in the United States. Removal of the coarser solids and the hair, by mechanical fine screens, eliminates not only several of the elements of unsightliness and trouble where tannery wastes are conveyed in sewers or streams, but also lightens, materially, the load on subsequent units of treatment, where required. Screenings are mechanically handled and placed in convenient receptacles for final removal.

Tannery wastes contain (as Messrs. Howalt and Cavett show) large quantities of solids. These are mostly organic and, therefore, highly putrescible, and their reduction is desired at the least cost. Sedimentation alone (3 to 4

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1675.

hours' detention period) will usually remove 75% of these solids. The addition of chemicals will add to this percentage, so that, if necessary, a clear effluent may be procured.

Clarity, however, costs money and all phases of a problem should be investigated before it is declared necessary to produce a clear effluent. Color is frequently hard to remove, at least to the point of invisibility in the effluent. After all, why remove it to that extent if the dilution factor of a stream is so high as to dissipate color when the effluent is mixed with it? Nature provides a decolorizing agent in the stream and it should be used.

Sedimentation basins have been developed, which eliminate the difficulties of older types. Manual handling of sludge, dewatering to remove it, duplicate basins, and deterioration of effluent by accumulating sludge, all have been eliminated by the use of the modern, mechanically cleaned, sedimentation basins, which have been successfully used at a number of tannery waste treatment plants.

Sedimentation alone will not remove color or bacteria. If their removal is required, other methods of treatment will be necessary. The work being done by the Joint Committee of Tanners and State Health Officials will undoubtedly add much to the literature of this subject and will provide an answer to the particular problem to be solved in Pennsylvania. As to its application elsewhere, the use of whatever remedy is finally decided upon as a result of these tests should be tempered with good judgment and careful study in order to make sure that that which is best for the client is recommended. The best for the client means the most economical form of treatment from a standpoint of first cost and operation cost, that will meet the requirements of the compelling authority. More than this is unequitable; less is prejudicial to health. Let the good work of experiment continue, but let engineers apply the results with good judgment.

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NEW THEORY FOR THE CENTRIFUGAL PUMP

Discussion*

By Elmer G. Hooper, Assoc. M. Am. Soc. C. E.

Elmer G. Hooper,† Assoc. M. Am. Soc. C. E. (by letter).‡—Consideration of the several different theories of pump design advocated and successfully used by recognized authorities in this field of engineering would lead an observer to believe that there must be more in common among the theories than is inferred in this paper. Pumps have been designed, built, and operated according to each of the different theories, and the mere fact of continued faith in any particular idea is evidence enough of a reasonable degree of success in obtaining desired results. The only satisfactory explanation for similar results from apparently different methods is difference in interpretation of design and test factors, and in the manner of co-ordinating them.

The author's statements, that a centrifugal pump may be considered as a series of three orifices through which all the water is to flow, that each orifice requires a difference, or drop, in head equal to the issuing velocity head beyond the arbitrarily assumed locations to produce the velocity; and, finally, that the sum of these drops is to be subtracted from the static head at no flow to give the resultant head per pound of water delivered, raise questions in the writer's mind as to logical interpretation of evidence.

There can be no doubt that the formulas representing the "new theory" gave convincing results when applied to the pumps designed and tested, and that they would undoubtedly do so with many others. However, the author's explanation that difference in pressure head equal to the delivered velocity head is needed in order to create such a velocity overlooks the fact that velocity through any opening, either orifice or nozzle, is the result of the combined velocity-of-approach head and pressure-difference head. Certainly there was an appreciable velocity of approach to the so-called

* Discussion of the paper by A. F. Sherzer, Assoc. M. Am. Soc. C. E., continued from January, 1928, *Proceedings*.

† Asst. Prof., Civ. Eng., New York Univ., New York, N. Y.

‡ Received by the Secretary, December 12, 1927.

orifices compared with the velocity of delivery from each one. The final interpretation by the author that these "drops" in head should be deducted from the normal static head at no flow to determine the delivered head puts a different light on the problem. A simple statement that "the loss in head through the pump is equivalent to the sum of the velocity heads mentioned", would be more effective and would give less cause for controversy. It is a substantial contribution to the common store of knowledge, however, if this seeming fact can be more thoroughly analyzed and more amply verified. The writer does not find sufficient data in the author's paper to analyze and verify the "new theory", certainly not enough to support the contention that all present theories are wrong.

Whether or not a centrifugal pump will produce a head equal to $\frac{U}{g}$ or $\frac{U^2}{2g}$ per lb. of water delivered, may never be settled to the satisfaction of every partisan, but either value may be obtained under proper conditions. In a pump with a circular casing and at no discharge there is likely to be a circumferential velocity producing a velocity head closely approaching $\frac{U^2}{2g}$ and a pressure head of $\frac{U}{2g}$. A piezometer at the circumference would register a head equivalent to the latter value; a Pitot tube inserted through the rim against the stream would register the sum of the two values, $\frac{U^2}{g}$. How much of this last value is available for delivery depends on the losses necessarily following transformation of this high circumferential velocity into the velocity in the system. For radial tip vanes, carefully designed and built, and with pump outlet designed to convert volute velocity gradually into discharge pipe velocity, a lift greater than $\frac{U^2}{2g}$ can reasonably be expected.

In any case the energy imparted to the water to lift it and to overcome all friction incident to the delivery is from work done by the impeller in its rotation. The energy equation would have to be for the whole system, including pump losses, pipe losses to and from the pump, sudden changes in absolute velocity wherever it occurred, bend and gate losses, the useful lift, and the work done by the pump. There may be some question as to the most accurate form in which the work done by the impeller may be expressed but, in the final analysis, the only difference is in form.

A constant speed pump can be really designed only for one fixed condition and output. When all passages are open and water is flowing smoothest, more energy per pound will be delivered for the work put in than under any other condition. To reduce the flow it is always necessary to close down on a gate in order to add friction or resistance to flow that the pump itself fails to offer. Every pound of water delivered then has to lose the additional friction head purposely introduced by the gate. The apparent efficiency of a pump at maximum is very much influenced by the design of the system, of which it

is a part, and the efficiency curve by the relative importance of gate friction to the whole. The author cites* an effective lift of 134.6 ft. at 200 gal. per min. and an effective lift of only 98 ft. for 800 gal. per min. For full range of operation in one setting, delivery head must be fixed at 98 ft. and for all flows less than the maximum the gate must serve to provide, directly or indirectly, friction loss or waste energy equivalent to the difference between the 98 ft. and the effective lift corresponding to that discharge. For example, in order to reduce the flow from 800 gal. per min. to 200 gal. per min., the gate must be closed so that every pound of water passing will lose 36.6 ft. lb. of energy.

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1792.

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WATER-POWER APPRAISALS

Discussion*

By Messrs. Joel D. Justin, Dana M. Wood, and Franklin H. Davis.

Joel D. Justin,† M. Am. Soc. C. E. (by letter).‡—There is literally nothing that has an intrinsic value. Even a mountain of gold if located in the midst of a desert has no value to its owner until marketability is assured. Certainly, an undeveloped water power has no value inherent in itself. The water power is merely the supply and there must be a sufficient demand before there can be even a presumption of value. Even then the water power would have no value unless it could be profitably developed. In thoroughly developed industrial sections there are many undeveloped water powers which have no value for the simple reason that no profit could be derived from their development at the present time or at any time in the future as far as can be foreseen.

The elements of value change materially with the development of industry and with developments in the art of utilizing water power. Thus, some of the high prices paid for water powers in New England mentioned by Mr. Cushman§ were for small water powers built several generations ago on tributary streams. In that era of hydro-mechanical developments the small streams with their inexpensive dam sites just suited the needs of individual mills. In general, nobody wanted the water-power sites with their expensive dams on the larger streams and these streams, therefore, had no value. To-day, the cost of operation, electrical equipment, and transformation, work against the utilization of the small powers and in favor of the large ones. To-morrow, perhaps, the development of the art will again make economical the development of small plants and the power sites on the tributaries will again have a material value. It is the present value or, at most, the prospective value

* Discussion on the paper by William H. Cushman, M. Am. Soc. C. E., continued from January, 1928, *Proceedings*.

† Hydr. Engr., U. G. I. Contr. Co., Philadelphia, Pa.

‡ Received by the Secretary, December 19, 1927.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1844.

in the near future which the engineer charged with valuing a water power, should consider.

Mr. Cushman evidently does not think very highly of the "Steam Substitution Method" of valuing water powers. Nevertheless, it is this "Steam Substitution Method" or "Competing Steam Plant Method", as it is called by many, which, taken in conjunction with the "Capitalized Net Earnings Method", is constantly used by public utilities to determine the advisability of purchasing or developing water power sites.

Mr. Cushman states* that, "the two blocks of differently produced power are not equal and cannot be made equal". The writer believes that Mr. Cushman is merely emphasizing the fact that the two blocks of power are not necessarily equal. They may be equal and frequently are, in the sense that the proposed hydro-electric plant will perform the same function as an additional steam plant. This is dependent on the position on the load curve which the proposed hydro-electric plant is able to assume.

In a usual case a power company, because of the growth of its load, is faced with the necessity of increasing its installed capacity. It can either add additional steam capacity or additional water capacity. A load study of the system is made and load-duration curves are plotted. At the same time a power-duration curve for the proposed hydro-electric plant is produced. From the load study it develops, perhaps, that the required additional capacity would have to operate, say, 20 hours during a peak-load week; in other words, the additional capacity would operate at very low load factor. From the power-duration curve it is determined that, during time of minimum stream flow, there is enough water available to permit the hydro-electric plant (with the available pondage) to operate at the required capacity during these 20 hours of the peak-load week. Hence, in this case, the proposed capacity for the hydro project is firm capacity. That is, the hydro-electric plant actually performs the same function as the alternative steam plant. The fact that the steam plant might produce additional energy during the other 148 hours of the week, when there is no demand for this energy, is a matter of no interest. During times of high water the hydro-electric plant, of course, produces additional energy which replaces energy in the base of the load curve. This energy would otherwise have to be generated by the existing steam plants.

Thus, the power company must choose whether the additional capacity will be in steam or water power. The cost of capacity and energy from the alternative steam plant is usually quite well established. These figures are applied as a yardstick to the proposed hydro-electric plant and if the cost of energy and capacity is materially less than from the steam plant the construction of the hydro-electric plant is indicated as advisable. The capitalized difference between the annual costs at the two plants may be taken as the ultimate value of the undeveloped water-power site to the power company; that is, if the company had to pay that sum for the land and riparian rights, the power and energy from the hydro-electric plant would cost just as much as the power and energy from the steam plant. Actually, the power company would not buy the water-power site unless it cost less than this. For this reason

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1843.

the power company must have a profit to compensate for the additional uncertainty of the cost of hydro-electric plants as compared with steam plants and because the capital investment required is usually much greater for water power than for steam per unit of capacity.

DANA M. WOOD,* M. AM. SOC. C. E. (by letter).†—As long ago as 1912, H. P. Gillette, M. Am. Soc. C. E., presented a comprehensive paper entitled "The Appraisal of Water Power Rights",‡ amplifying another excellent contribution by Charles T. Main, M. Am. Soc. C. E., entitled "Computation of the Value of Water Powers and the Damages Caused by the Diversion of Water Used for Power".§ Other good papers might well be mentioned, but the foregoing appear to the writer to be the best statements of the several theories most commonly used.

By definition, valuation is "the act of valuing" or "estimating the worth or price"; appraisal is "the act of appraising or putting a price upon with a view to sale". In practice, the two terms are frequently used interconvertibly. Value and worth ordinarily apply to "purchase" or "sale value", but there are other values, such as investment value, tax value, and value for rate-making purposes. Therefore, because of this incompleteness in the terms, "valuation" and "appraisal", it is recommended that the qualification as to purpose be added, such as "water power appraisals for sale purposes".

If sale value is the purpose of the "valuation" or "appraisal", it is not necessary, often, to go to the expense of making an inventory and an appraisal of the physical property, because sale value is usually best determined by ascertaining what the property will earn. When the appraisal is made in such cases it is rather for the purpose of determining the condition of the property and the amount of money necessary to restore it to full service value.

It would be very helpful to have available a reliable list of actual sale prices of properties involving water rights, but most business men do not desire to disclose such data. If used properly, the report of the New England Water Works Association, referred to in the paper,‖ is valuable as an index, and is the nearest approach to a general compilation of past values now available.

The author's method of analyzing stream flow and estimating power available should not pass unchallenged. Several recent papers indicate more modern methods. In March, 1916, the writer presented a paper in the *Journal* of the Boston Society of Civil Engineers, which several engineers discussed in the June issue. Several subsequent papers on these matters have been presented before the Society; and recent textbooks have included worth-while discussions. A large Committee of the Boston Society of Civil Engineers, known as the "Run-off Committee", presented a report which was published in October, 1922.

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* Hydr. Engr., Stone & Webster, Inc., Boston, Mass.

† Received by the Secretary December 20, 1927.

‡ *Engineering and Contracting*, Vol. 38, p. 624, December 4, 1912.

§ *Journal*, New England Water Works Assoc., Vol. 31, p. 213, September, 1907, and *Transactions*, Am. Soc. Mech. Engrs., Vol. 26, p. 68, 1905.

‖ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1844.

In Mr. Cushman's paper unusual definitions are set up for "primary" and "secondary" power. To avoid confusion, it would be preferable to use "day load" and "night load", or some other similar terms. With so much already in print, it is necessary only to call attention to the shortcomings of the paper in these particulars.

Property values along with other items of cost determine the rates charged, and the rates charged affect the gross earnings. Courts and commissions alike invariably object to any method of predicating value on gross earnings. Herein lies the most confusing feature of the paper, since market values of power are predicated upon underlying values, including land and water rights at hydro-electric plants. Furthermore, the author uses an average market price of power, and any individual plant should receive any benefit, or conversely be penalized, from favorable or unfavorable factors of location, etc.

To the writer there appears to be one, and only one, correct way of arriving at a proper valuation of any water right. The first step is to arrive at a value—not necessarily the finally adopted value—by the method generally known as the "capitalized net savings when compared with the next available source". (Occasionally, it is necessary to consider using the water power for an entirely different industry.) The next available source of power may be steam, central station service, another hydro-electric plant, etc. This value should always be called "the maximum limiting value". It simply offers a yardstick or measuring device by which the finally adopted value can be reached. In a sale this is by barter and trade. In a tax case it is suggested that the procedure followed by local assessors, that is, fair percentage of the maximum limiting value, can be taken by agreement.

The author's method is really a special case under this general statement, because, if the next available source is a public utilities power line, the cost of power purchase from that company—it might well be at wholesale rates—becomes the standard of comparison. In such a case, however, the annual and operating charges on cost of interconnection and changes or additions required at the point of delivery to permit using this form of power, must be added to the purchase price for the power.

Value is never a fixed or certain amount for all time, but changes from time to time with certain fundamental elements. Sometimes several different values as of a given date can be obtained by very reasonable variations in the basic assumptions, estimates, or analysis of conditions, and engineers may accordingly differ somewhat in their opinions as to the value of the same property.

General Principles.—A "horse power" has a value because the power of which it is a standard of measure can be made income-earning, just the same as land located at a street intersection. A corner lot is more valuable on account of its location than the lot adjoining, although of equal area, because it has greater street frontage and greater possibilities of development and, therefore, a greater earning power. This income-earning possibility is the measure of the value, and the value is that sum capitalized which will bring the income at a fair rate of interest.

The value of a horse-power is determined by a detailed calculation of its earning power. It naturally has no fixed market rate, and never can have. Each power privilege must be treated separately and on its merits. A large power site located in an industrial center will always command a high value, while the same site situated in the backwoods, miles from anywhere, might be unsaleable.

The principles for determining the value of a water right may be definitely fixed, but no set rules can be formulated which will be applicable to all conditions and situations. The principles may be retained, but modified to suit individual cases.

In determining the value of property of a public utility for rate-making or other allied purpose, the public interest is directly and vitally concerned. Some Courts and commissions have held, in their decisions on rate-making or capitalization, that, in the case of such a natural resource as water power, the public is entitled to at least some part of the benefit of the comparatively cheap power from this source in return for its practically guaranteed protection of the public utility from all unjust or unreasonable demands, the granting of the right of exercise of eminent domain, and the granting of practically monopolistic control of the market for the output of the power station.

On the other hand, valuation for the purpose of purchase and sale of property between private individuals or corporations is not subject to the same regulatory control. The seller is interested in securing the full value of his property on the basis of what it has been and is expected to be worth to him as an income producer, while the purchaser is governed by consideration of what the property will be worth to him as producing an expected income. The price is usually reached through bargaining. This is the real reason why the capitalization of earning capacity is the only direct and proper method of valuation of a natural resource like water power. The personal equation is here present, as it is in all transfers of industrial ownership, since people do not all think alike, and this should accordingly be recognized in any disinterested appraisal.

The foregoing shows the necessity for a clear distinction between ownership, subject only to the general laws of the country and ownership subject not only to the general laws, but also to the regulatory laws, of public commissions.

Valuation of water rights based on so-called market value, depending for its establishment on records of sales of similar property, cannot with any degree of propriety be applied to a natural resource, unless checked by a consideration of capitalized earning capacity. Such a check would be necessary to demonstrate that the sales for comparison were made at the actual face value of the property, and not at a price either above or below it, as has often occurred.

So-called market value is ever changing as a result of the laws of supply and demand or competition, and is subject to recurring periods of activity and depression, rising and falling prices, and production costs. It is possible that value for condemnation is the nearest approach to exchange or true economic value.

While value for sales has been emphasized in the discussion, that for other purposes may be obtained by modification of some of the items considered in determining sale value. Then, too, valuations for sale are more frequently made than for any other purpose, and this justifies setting forth the principles of such a valuation first and then studying the necessary modifications for other purposes.

The U. S. Supreme Court has ruled (C. C. C. & St. L. R. R., *vs.* Backus, 154 U. S. 439), that:

"The value of property results from the use to which it is put and varies with the profitableness of that use, present and prospective, actual and anticipated. The amount and profitable character of such use determines the value."

The value of a water power privilege may be treated under three general cases: (1) Undeveloped water-power site; (2) completely developed water-power site; and (3) partly developed water-power site.

Undeveloped Site.—The owner of the site does not own the water, but merely a right to use it in a reasonable manner without serious interference with the similar rights of others on the stream. (There are many interesting legal questions and situations involved in this simple statement and the many correlated questions raised, which are not included in the scope of the paper.) This right has some value, because it is possible to buy the particular land including this right for the privilege of making use of the power that can be developed. As the privilege stands undeveloped, it is non-income earning, but it certainly has a future or speculative value. The value is affected by considerations of geographic position, relative location to point of possible use, and future possibilities. On this latter point, expert opinion may vary, and probably rightly so, because men cannot be expected to look on futurity from the same viewpoint. The problem is similar to that of corner lots: One may be on a street with good future prospect for improvement, while the other may not be so favorably located.

Power can be generated and transmitted long distances, and it is not necessary to use it at the point of generation. The cost of necessary transmission, however, reduces the value of the power site in some ratio proportional to the distance. Similarly, the location of the site may be such that the cost of transporting the raw and finished products of the business, if located directly at the site, may be so great as to make the water power uneconomical, and, therefore, valueless. The paper by Mr. Main, and the other papers previously cited, furnish the general methods of procedure to follow in actual cases.

An undeveloped site may fall under one of several classifications:

1.—When the riparian lands, subject to submergence by the construction of the dam, are owned by several parties not necessarily in harmony. Under this condition, no complete development of power is possible and the value of the particular riparian rights, bearing their proportion of the value of the entire group, must be discounted to the extent that moneys must be spent to obtain consolidation of rights.

2.—When all the riparian lands have been consolidated under one ownership, making a complete development possible.

3.—When that consolidated ownership has also the financial ability and resources to make the development, and the business ability and opportunity to create a market for the power.

Fully Developed Site.—The value of a developed horse-power is determined from its demonstrated earning capacity resulting either from the sale of power or from its advantageous use in manufacturing processes, as represented by the saving obtained therefrom when compared with the cost of equivalent power from the next cheapest source or at another locality.

The difference between the annual cost of a horse-power produced by a water-wheel and by the next available cheapest source of power, often steam-generated, is the saving due to the use of water power, and this sum, capitalized at a fair rate of interest, will represent the maximum limiting value of a horse-power at that particular water privilege.

The size and character of the power to be acquired from some other source should correspond to the power utilized, should be of a class suitable for replacing the water power, and should be of an economical type. If transmission is required, or extra charges must be paid on account of a different location of the industry, proper allowances must be included in the comparison. The question of stream flow must be properly considered, and if an auxiliary steam plant or storage development is necessary proper allowances should be made. Each case must be treated by itself.

In determining value by capitalization of demonstrated or anticipated annual earnings, or of actual or indicated saving, it is necessary in computing either profit or saving to recognize that it is not attributable in its entirety to the water-power right, but that a part of such profit or saving has been actually earned by the invested capital represented by the cost of the dam, waterways, power station building and its equipment, and transmission lines to the point of usage, also cost of necessary right of way. This part of the profit or saving should be considered as interest on the corresponding capital investment, at the rate usually sought in the particular industry, and is properly deductible from gross returns, together with other operating expenses, to obtain the net used for capitalizing.

Partly Developed Site.—In this case there may be two values to be combined, determined as for each of the previous two cases—a real value plus a speculative value.

Worth of Developed vs. Undeveloped Site.—A question frequently raised is, "Are the lands and water rights worth as much at an undeveloped site as they are at a developed site"? The potential worth is the same in either case, but, as suggested in the foregoing, future opportunities for earnings are not considered the same as the demonstrated earnings of a well-established business. One does not have, and the other has, a going concern value. The same development steps are ultimately necessary to change the undeveloped to a developed site. The enterprise must be promoted, organized, built, and started, all at a cost.

The one essential difference in the two cases is the item of "contingencies" in the sense of the unknown or unforeseeable factors that must be met during the development stage. In the case of the developed site, these factors are usually a matter of record, susceptible of analysis and more definite valuation than in the case of the undeveloped site.

To allow for the differences in the two cases, it is quite customary to use higher unit costs in the estimates for the undeveloped site, as well as a higher percentage allowance for contingencies. Frequently in addition to that mentioned, another method is to use a higher capitalization rate for the undeveloped site.

Provided no factors have been omitted in the estimates, the differences in expert opinion regarding the value of a particular site are largely due to differences in estimated unit and item costs, contingency allowance, and capitalization rate.

The net result is usually that a developed site is considered to be more valuable than an undeveloped one. In this connection, however, it should be remembered that advances in the art of engineering have rendered some old plants obsolete, and the cost of removal or alterations to modernize may offset the difference in riparian value between the developed and undeveloped site. This is reflected in the comparative cost estimates or appraisals.

Controlling Factors.—The principal factors governing the value of a water power site may be summarized as follows:

1.—The Characteristics of Stream Flow: The flow of no two rivers is identical. No two power sites have the same available drainage area, geological or topographical conditions, or precipitation and run-off conditions. A study must be made of each case.

2.—The Power Head Available: Topographical conditions largely govern this. While the power theoretically is directly proportional to the head available, the cost of the development is not. There is no fixed cost relation between sites.

3.—The Stream-Flow Use: One power development may require a steady continuous flow day by day, while another may need intermittent use and the creation of pondage to conserve water. The utilizable stream flow may come in such irregular manner as to require a storage development or a relay plant of some kind, which must be included in any comparison with power from another source.

4.—The Cost of Development: This is variable, depending on, not only location and size, but material and labor costs, and other factors.

5.—The Character of the Market: The availability of a market for the power produced; possibilities of increased use, of present and future competition; the hours of use and variations in hourly, daily, and seasonal demand, are all important factors.

Where a manufacturing plant can develop a water power and utilize the energy at less cost than it can obtain a similar amount of power and energy from other available sources, it is contended that the value of the water power to the industry may be gauged by capitalizing the net saving in comparison with such other sources of power. In the event that the manufacturing business is discontinued, the value of the water power may then be determined by consideration of a new use to which it can be put. Water rights in the latter event might have only a fraction of the value in the former case.

Increased manufacturing cost resulting from locating at a water-power site, such as increased haulage, freight, higher labor costs, etc., should be charged against the water power.

The maximum limiting value of water rights is obtained by using normal interest rates, insurance, taxes, and similar items and average stream-flow figures. On the other hand, a minimum limiting value may be obtained by using a more conservative basis for the various estimates and the run-off available in the driest known year. Engineering experts may fairly differ in their evaluation of these various factors.

It occasionally happens that a certain manufacturing plant is located where water power may be developed cheaply and where the cost of steam power or other next available source would be so excessive that a deficit on the sale of manufactured products would result with the substituted power. In such a case it would be futile to value the water rights on a full comparative basis, but rather it should be considered that the entire net profit of the business capitalized would represent the value of the water rights.

It should be noted that, if interest is not included in the annual operating costs, it is necessary to subtract the physical cost of the plant from the total value obtained by capitalizing a net saving when compared with another source of power, in order to obtain the value of the land and water rights. On the other hand, if interest is included in annual operating costs, the value obtained is that of the land and water rights. In either case, careful consideration should be given to the elements which justify the selection of the rate used in capitalizing. The item of "cost of money" should be considered in particular. Some physical plant cost estimates include all overhead items; others do not; and this question largely influences the selection of the rate of capitalization to be used. The author is not at all clear as to the background for the 10% rate he suggests.*

As pointed out by G. F. Swain, Past-President, Am. Soc. C. E., in "The Principles Governing a Railroad Appraisal of a Universal Nature",† the purpose for which the appraisal is made affects materially the method of handling some of the items of cost.

The writer does not claim originality in the foregoing remarks, because much has been obtained from the work of others and put together to substantiate his opinion that Mr. Cushman's paper, not only offers nothing new in principle, but makes statements and suggests methods which to-day should not be considered as good practice, or should be enlarged on in explanation.

Franklin H. Davis,‡ M. Am. Soc. C. E. (by letter).§—The author discusses several methods sometimes used in the appraisal of water-power properties and calls attention to some of the inconsistencies of the so-called "Steam Substitution Method." It is claimed that the "Real Estate Sales Method", based on current and comparable sales of water-power property, is unreliable,

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1850.
† Abstracted in *Engineering Record*, Vol. 65, p. 174, February 17, 1912.
‡ Valuation Engr., Bureau of Internal Revenue, U. S. Treasury Dept., Washington, D. C.
§ Received by the Secretary, January 9, 1928.

but that the "Capitalized Net Earnings Method", as outlined by the author, will produce logical and consistent values if the factors are correctly used.

In discussing the Steam Substitution Method, it is very properly pointed out that a method of valuation that is not in good repute with the Courts is not available to the appraisal engineer. On the other hand, the Real Estate Sales Method is admittedly accorded full attention by the Courts, and, therefore, it would appear by the same line of reasoning that it should have some standing with the engineer.

The reason assigned for the full attention given to the Real Estate Method is the alleged disinclination of the Courts to study data of a "highly technical nature."* It is believed that the Courts have in many cases accepted the Real Estate Method as the most reliable basis of valuation offered to them in lieu of other method involving calculations based on theoretical and uncertain assumptions, which calculations have often been involved and indefinite, rather than highly technical.

The writer agrees that the awards listed in the report of the New England Water Works Association, mentioned by the author,* have little or no value as sales transactions for several reasons, in addition to the one mentioned, but he does not believe that these particular cases serve to show the entire unreliability of the Real Estate Method.

There are so many varying situations in a water-power development with respect to quantity of power, its constancy, reliability, location, and availability for different uses, that it is believed unsafe to apply any general rule or formula for the appraisal of all such properties. The author advocates a method of capitalizing the estimated hypothetical net earnings under the assumption that the power is sold at current market rates. This assumes something that would have to be proved in each case and which is probably not true in most cases, namely, that a market exists for exactly the block of power available, subject to the limitations and uncertainty of the output from a water-power plant. In order to produce a salable block of power there would be required in many situations a steam auxiliary, or other source of reserve power, and transmission lines to reach a market assumed to exist. As a matter of fact, single water-power plants in the United States, with a few notable exceptions, can not be operated by themselves as paying propositions, in the selling of electric energy under the standard forms of contracts.

When a water power, either mechanically or electrically developed, is an integral part of a manufacturing operation, it should not ordinarily be separated from the other component parts of the operation and assigned a separate value. Some industrial processes require cheap power. Such a condition would frequently result in increasing the value of one part of the property—the water power—at the expense of the remaining part. The value of the part remaining might be practically destroyed in case the industry could not operate at a profit when forced to buy power at the current market rates used in measuring the value of the water power. If market values are established at all, it is believed they are more likely to be indicated in the partic-

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1844.

ular industry using the power, and by the earnings actually realized in that industry, rather than by an estimate of earnings that might be made if the power had been used in some other kind of business.

Current market rates for electric energy in many localities reflect to a great extent and are often predicated entirely upon the cost of steam-generated power. The method recommended would in effect introduce and apply, by indirection, the same erroneous element of comparison—the Steam Substitution Method—which the author has very properly disclaimed.

It is not contended that the Real Estate Method should be a compulsory one simply because it is looked upon with favor by the Courts, but that in most cases it should not be ignored and should be considered along with all other evidence available. Some of the important factors which might have a bearing on value, and which it is believed should be considered, are: (1) The actual cost of developing the power and the date of its development; (2) the cost of securing lands and site; (3) the cost of reproducing depreciable structures; (4) accrued depreciation; (5) the availability and cost of similar sites in the same territory and the cost of developing them; (6) the actual earnings in cases where power is developed and sold; (7) the actual earnings in the business or industrial operation where the water power is a component part of the plant and property utilized in the industry; (8) stream-flow characteristics at the site; (9) amount of primary and secondary power available under average conditions; (10) purchase and sales transactions, in similar and comparable properties; (11) capital cost per horse power for the actual development of similar properties; and (12) rentals of water power. These and other subjects are all proper matters for consideration by the appraisal engineer. It is believed to be impossible to lay down any formula or method applicable to all cases for the valuation of a water power, except that of carefully weighing all the facts in each particular case and determining in the light of those facts, and by the exercise of good judgment, what a property is worth on the market.

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THE GREAT YANGTZE BAR

Discussion*

By R. L. Vaughn, Assoc. M. Am. Soc. C. E.

R. L. Vaughn,† Assoc. M. Am. Soc. C. E. (by letter).‡—Under the conditions outlined in this clear and interesting paper, any one experienced in problems of this kind would probably agree that the most promising line of endeavor would be by dredging. Training works appear to be almost out of the question: First, on account of their magnitude and cost; second, because of construction difficulties; and, third, because of doubts concerning their efficacy and permanence. Quite often under such circumstances jetties merely push the problem a little farther seaward, so that, with improved equipment available, the tendency in the United States has been more and more to rely on dredges to create and maintain channels over ocean bars.

Hydraulic hopper dredges are best adapted to a sandy material. With silts, from 5 to 10 yd. are often pumped overboard for every yard retained in the hoppers. The net results on the channel will be excellent, however, provided there is a current to carry away the material thus placed in suspension. Using a certain special type of suction head, remarkably satisfactory results have been obtained in moderately stiff mud. Where the distance to the dumping grounds has been relatively short, costs as low as $0.03 per cu. yd. (hopper measurement) have been achieved, this figure being exclusive of capital and administrative items.

With anything approaching a moderately stiff clay, so far as the writer is aware, drag-suction equipment has been a miserable failure. It hardly seems possible, in a location consisting of shifting mud flats, that there should be any trouble from this source. However, the material is described as being very firm, although it cannot be classed as a real clay. It is further stated that considerable resistance is presented to grab or bucket dredges. A

* Discussion on the paper by H. von Heidenstam, M. Am. Soc. C. E., continued from January, 1928, *Proceedings*.

† Cons. Engr., San Francisco, Calif.

‡ Received by the Secretary, December 20, 1927.

first-class bucket dredge is capable of excavating very hard material. Any substance which presents a marked resistance to such a tool might well prove impossible for a drag suction.

There seems at least a possibility that the pipe-line dredge has been dismissed without giving it sufficient consideration. The outstanding objection to the use of such equipment in the present instance is the exposure to rough water. It is quite true that a pipe-line dredge could not be operated in such a location satisfactorily if held in position by spuds. It might be entirely feasible, however, to dig on wires.

Another difficulty would be with the pipe line itself, which would need to be of special and rugged design. So far as the writer knows, no equipment which would exactly meet the present condition has ever been built, for the reason that it would be quite expensive and that there was little or no demand for it; but there is no evident difficulty that could not be overcome.

The author states and the writer agrees that, other things being equal, such a tool would be preferable. One important possibility is that the material, if it is anywhere near as stiff and sticky as the description indicates, would pile up at the end of the pipe line to a considerable extent, thus forming a bank alongside the channel. Such a bank would be subject to attack by wave action, but, nevertheless, it would remain a very distinct shoal, and tend to perform the function of a training wall, for a considerable period.

One of the great advantages of a pipe-line dredge is that when it is working at all it is working continuously. Except when interrupted by serious storms, a properly designed plant under the given conditions should be able to work 22 actual pumping hours per day, 25 days per month (if shut down on Sundays). The author states that the weather is moderate and the water calm 80% of the time throughout the year. If there should be a 20% loss of time due to rough weather, this time could be partly used for plant repairs and maintenance, giving an actual pumping time of 440 hours per month, or about 5 200 hours per year.

A dredge of the seaworthy characteristics required could easily be designed to have a capacity of 4 000 cu. yd. per hour, giving a capacity of 20 000 000 cu. yd. per year. Such output may appear fantastic to one not familiar with the latest developments in plant of this kind, but it is not much beyond the range of the writer's own personal experience. For instance, he has seen 2 000 cu. yd. per hour pumped through 1 500 ft. of floating 22-in. pipe line, the material being a mixture of 60% stiff mud and 40% broken oyster shells, which is a combination not particularly easy to pump. This particular dredge had about 1 200 h. p. (900 kw.) on the pump shaft.

Even should the capacity be grossly over-estimated, due to lack of familiarity with the material and the local conditions, still two such units would certainly be able to greatly exceed this annual output, if they could be used at all. A unit such as is contemplated, together with all its attendant auxiliaries, could be built in the United States for a cost probably under $1 000 000. In this country the operating cost of such a unit, exclusive of capital and administrative charges, but including maintenance and renewals,

should not exceed $50 000 per month. These expenditures, both capital and operating, are materially below those given by the author.

With such an increase in the dredging capacity possible, much of the force would be lost from the argument that the cut should be deepened gradually and uniformly throughout its entire length. Of course, there would be considerable silting in the cut, but probably not enough to affect seriously the problem. Certainly, also, a part of the material pumped would find its way back into the excavated channel, but not as great a part of the actual quantity pumped as would be the case with a hopper dredge.

Many practical details of such a dredging plant would require special study for solution. One of the most important elements would be the pipe line and the method of controlling it. The line should have as little excess buoyancy as possible, as one with a large excess of flotation would pound itself to pieces in a seaway. The effect of the strong currents on the floating line should also be carefully considered; likewise the method of holding and moving it. Doubtless some of the floats would go aground on the spoil-bank, but with the large range of tide the line should float at high water and could be shifted ahead at that time. The moorings to hold a large dredge on a mud flat should be given special consideration; likewise the means for shifting—always remembering that these operations must be carried on regularly in rather rough water. For anything like satisfactory running time it would be necessary to re-fuel while dredging, which would probably preclude the use of anything but oil fuel. It is suggested that, for power, first consideration be given to a direct-connected Diesel drive for the pump, and Diesel-electric drive for the cutter, swinging engines, and auxiliaries.

The writer has been encouraged to advance his present suggestion by a consideration of the depths of water that seem to exist in the vicinity. A greater part of the area over which it is desired to dredge appears to be protected somewhat by a rather extensive stretch of comparatively shallow water to seaward. Under such conditions a very nasty choppy sea may exist, one quite dangerous to small craft, but different from the heavy destructive surf which may be encountered where there is deep water in the immediate vicinity of a shoal. If there should exist in the locality in question the heavy breaking seas to be found at times in such places as San Francisco Bar, Eureka Bar, or Humboldt Bar, on the Pacific Coast, for instance, then only a highly mobile plant, such as a self-propelled hopper dredge, can be considered. It might be that a combination of the two tools would give the best solution, a self-propelled dredge for use on the outer part of the bar where the exposure is greatest, and a pipe-line machine for use farther in where the expanse of shallow water had broken up the heavier seas.

In any case the advent of a typhoon is an entirely different matter. The approach of such a storm can be detected, and retirement of all plant should be made to the most sheltered spot within reach. This would necessitate the services of a very able sea-going tug which should be available at all times. No dredge that the writer can imagine could successfully ride out a typhoon while lying on the bar.

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SIDE SPILLWAYS FOR REGULATING DIVERSION CANALS

Discussion*

By Edward S. Lindley, M. Am. Soc. C. E.

Edward S. Lindley,† M. Am. Soc. C. E. (by letter).‡—The author arrives at a coefficient of 3.82 for the spillway section shown on Fig. 6.§ This is probably about the maximum value of coefficient that it is possible to realize, as long as no negative pressures are induced at the control section; but tests made by the writer‖ indicate that that value is likely to be realized.

In the formulas and calculations of Table 2,¶ the author measures the head causing velocity down to the center of the orifice. That is the text-book method for a small orifice the jet from which discharges freely into the air all round it; but a different solution applies when the height of orifice is not small relative to its submersion up stream, and when down-stream water is above its bottom. Where, as here, side and bottom contractions are completely suppressed, and the jet from the orifice is supported on the sides and bottom, Bernouilli's theorem gives a simple rational formula, so elementary that no demonstration is needed.

Following the author's notation, but splitting his coefficient of discharge into:

c_v = the coefficient of velocity;
c_c = the coefficient of top contraction; and

h_a = the head to be added to allow for velocity of approach (actually greater than $\frac{V^2}{2g}$)

$$Q = c_v\, c_c\, d\, \sqrt{2\, g\, (d_1 + h_a - c_c\, d)}$$

* This discussion (of the paper by W. H. R. Nimmo, Assoc. M. Am. Soc. C. E., published in October, 1927, *Proceedings*, but not presented at any meeting) is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Executive Engr., Indian Public Works Dept., Panjab Irrig. Branch, London, England.

‡ Received by the Secretary, December 22, 1927.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1880.

‖ *Paper 97*, Panjab Eng. Congress, 1925.

¶ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1884.

The simplest case is that of a bellmouthed orifice with c_c equal to unity. The writer has confirmed it in tests of a 6 by 3-in. orifice, under heads up to about 2.5 ft. Plotting the squares of discharges against the up-stream water level, the "curve" was a straight line which cut the axis of zero discharge at the level of the top of the orifice; the bellmouth used had a c_v of 0.975 over the range of heads tested.

In the sluices of the Assuan Dam* there is contraction, and c_c varies with the gate-opening, at any rate when the slightly belled tunnel opening comes into play with bigger openings. Discharge from the tunnels being freely into air, the head was measured to the center of the orifice, as in textbooks, and the resulting coefficient was found to vary with the head. At the writer's suggestion the rational formula was applied to the data, when the coefficient was found to be independent of the head, and the top contraction was calculated.

Where, as in the observations given by the author, the height of orifice is greater than the critical depth of flow, and probably also where it is only a little less than that depth, conditions are uncertain; they are even probably unstable, undergoing a slow pulsation.

The observations are too few for analysis; and it would seem that they are not full and certain enough to make analysis worth while. The author has measured his d_1 at the up-stream face of the gate, including banking up, and has still taken the whole velocity of approach into account. Velocities were so high that the difference between h_a and $\frac{V^2}{2g}$ would be appreciable.

* *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI (1922-23), pp. 213 and 257.

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ADVANCES IN WATERWAYS ENGINEERING DURING A HALF CENTURY

Discussion*

By Messrs. C. L. Hall, and De Witt D. Barlow.

C. L. Hall,† M. Am. Soc. C. E. (by letter).‡—The author has made what amounts to a survey of standard river and harbor practice. The special type of Mississippi River dredge described§ is not, and cannot be, used on the Upper Mississippi. The dredge used there is a pipe-line dredge, without cutter, and without propulsion machinery, and was first introduced in 1899.‖ The dredge is held by spuds, the position of the suction pipe being controlled by a catamaran,¶ moved by a swinging engine. The accordion type of joint, originally used, has now been replaced by a ball joint. The normal cut of the dredge is 80 to 100 ft. in width.

The Upper Mississippi type of dredge is used not only for obtaining requisite channel depths, but also for securing material for wing-dams, the two purposes being combined whenever possible. The "sand dams" were first developed during the season of 1911.** They are built out from shore, and usually are completed only to the elevation of low water. They are then capped by the ordinary brush mattress with rock cover. The sand dam is such an economical type of construction that, when dredges are available, it has almost entirely replaced the brush and rock dam along the Upper Mississippi, except in stretches where the slope is abnormally steep.

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* This discussion (of the paper by W. M. Black, M. Am. Soc. C. E., presented at the meeting of the Waterways Division, Philadelphia, Pa., October 8, 1926, and published in October, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Major, Corps of Engrs., U. S. A., Rock Island, Ill.

‡ Received by the Secretary, October 8, 1927.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, pp. 1943-1944.

‖ *Engineering-Contracting*, Vol. XL, No. 13, p. 361 *et seq.*

¶ *Engineering News*, July 24, 1913.

** *The Military Engineer*, September–October, 1924, p. 389.

DE WITT D. BARLOW,* ASSOC. M. AM. SOC. C. E. (by letter).†—This very interesting paper contains two tables of costs, which the writer thinks are likely to create misunderstanding unless they are further explained.

Table 2‡ gives the average performance of Government-owned "Hydraulic Pipe-Line Dredges, Cutter Type" during the years 1923, 1924, and 1925, and states the costs as 8.32, 8.48 and 7.44 cents, respectively.

Table 3§ ("Performance of Hydraulic Pipe Line Dredges Working Under Contract"), gives the average price per cubic yard for the years 1923, 1924, and 1925, as 20.99, 21.3, and 19.9 cents, respectively. While the casual reader might draw the inference that the cost of hydraulic dredging under contract is vastly greater than that done by Government plant, this is not the case.

In the first place, it will be noted that the total yardage on which the Government figures of approximately 8 cents are based, is about 1 500 000 yd. per year, a very trifling amount; whereas the contract yardages, from which the average prices are obtained, vary between 13 800 000 and 25 000 000 per year. So there is no reason to suppose that there is any similarity between the work done by the Government and by contract plant. Even, however, if the work were identical, the figures would not be susceptible of comparison for the following reason. The contract prices represent the total cost to the United States of performing the amount of useful dredging done. The writer uses the words "useful dredging" advisedly. Contracts are let by the United States for the excavation of channels of fixed widths and depths, and with certain fixed side slopes. The contract instrument provides that no payment will be made to the contractor for any work done outside the lines of the desired improvement. The contractor is required, however, to clear those lines, that is, to remove all material from within them. To accomplish that end, it is necessary to remove very large quantities of material from without the prescribed sections. This is called "no pay overdepth." It very often happens that in order to be paid for 1 000 000 yd. of excavation the contractor must remove 1 500 000 yd., or even 2 000 000 yd.

In the computation of costs of operating by Government plant, no account is taken of the amount of useful dredging performed. The amount of yardage set down as excavated is all of the yards removed by the dredge regardless of where they were. Even this determination is often made with the most trifling regard for accuracy. Sometimes the daily reports of the dredge captains are taken, which are no more than guesses, and are not likely to be too low.

The Government cost figures are obtained by dividing certain expenditures by the yardage thus determined. The expenditures themselves, however, do not include all, or nearly all, the cost to the United States of doing the work. They usually contain merely the operating cost of the plant while it is on the job. They never include plant rental, that is, interest and depreciation on the investment, which is a very considerable item; nor insurance (fire, marine,

* Pres. and Director, Atlantic, Gulf & Pacific Co., New York, N. Y.

† Received by the Secretary, November 11, 1927.

‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1950.

§ *Loc. cit.*, p. 1951.

and liability); nor do they include any of the costs of maintaining idle equipment.

When the United States rents its hydraulic pipe-line dredges to private parties, it charges 0.1% per day of the first cost of the plant for rental, that figure having been determined after a careful study of their real cost experience. That being the case, it is obvious that a dredge that costs $1 000 000 has fixed charges running against it in the amount of $1 000 per day, no account of which is taken in determining the nominal cost of Government operations. The upshot of the whole matter is, therefore, that the 8-cent costs are only costs in a very special sense. They are quite meaningless, unless one happens to know the formula. In order to compare these nominal Government costs with contract prices, it is necessary to make all the foregoing corrections, and others. Within the writer's experience the effect of these corrections is often to increase the cost to several hundred per cent. of the nominal one. It is astonishing what happens to a fraction when one continues to add to the numerator and to subtract from the denominator.

The writer can see no harm in keeping and publishing such nominal costs as long as the correlative circumstances controlling them are published or generally known. The difficulty is that they are not generally known and these data, which may be useful in the War Department, for purposes of its own comparisons, are likely, as in this case, to come before the public in an unexplained form and to mislead, although there is no intention on the part of any one to do so.

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BOUNDARY SURVEYS

Discussion*

By Messrs. R. H. Randall and W. H. Halsey

R. H. Randall,† Assoc. M. Am. Soc. C. E.—The speaker would like to add a word in defense of the ground map.

Referring to the relative value of air and ground methods, there is now a certain type of map that is different from the hydro-electric map or the transmission map, even though it may be plotted on the same scale. For the kind of map required in sanitary district work outside of cities, commonly on some such scale as 400 ft. to the inch, where the land is worth several hundred or several thousand dollars per acre, it pays to have all the planimetric accuracy obtainable.

Maps of that kind carry from five to twenty-five, or more, stadia locations per acre, and the topographer, if he is honest, neither overlooks any part of the terrain for which he is responsible, nor can he possibly miss the location of any prominent object. It is not fair to base any comparison of air and ground maps on the assumption that ground mapping is done with the chance of overlooking any essential information.

A point that was brought out by Professor Johnston and, also by some of the discussors, is the great need for control. Of course, it is desirable to have all monuments set exactly where they should be, to be as free as possible from interference, but the absolute importance of preserving them diminishes if they have been co-ordinated and joined to a controlling survey of adequate precision.

If triangulation and traverse were extended to the point that seems logical, there would not be so much trouble in New Jersey or New York, or the other States, in property matters, topographic matters, or in anything else that pertains to surveys.

Incidentally, one of the instrument manufacturers in this country now turns out a theodolite with optical qualities that permit of daylight observa-

* Discussion of the paper by C. T. Johnston, M. Am. Soc. C. E., continued from January, 1928, *Proceedings*.

† Pres. and Chf. Engr., R. H. Randall & Co., Inc., Toledo, Ohio.

tion of Polaris, which still further reduces the difficulty in obtaining an original bearing.

The State of Florida has no adequate level datum. It is impossible to run precise levels any place in the southern or central part of the State, because no controlling system of bench-marks has been established by the Federal Government; nor can this be started, apparently, before 1928. Yet there is an urgent need for levels there. The country is flat, and there is every reason why all the engineering that is going on should be tied in to precise elevations. The cities and other political divisions, and private projects must suffer for 1½ or 2 years, perhaps, before they can even begin to get the level data they need for engineering work.

While there is some evidence of the appreciation of control work, by cities and counties, there does not yet seem to be sufficient appreciation and support for Federal control surveys.

The expectation of rapid acceleration of the programs of the U. S. Coast Survey and the U. S. Geological Survey under the Temple Bill have not been realized, chiefly through lack of Federal and State appropriations. The furtherance of the vitally important work of these two surveying agencies of the Government is a matter of immediate concern.

W. H. HALSEY,* ASSOC. M. AM. SOC. C. E.—Attention has been called to the standardization of tapes. This is certainly important. An engineer cannot retrace a survey with confidence after a period of years unless it was measured accurately in the first place. Monuments will become displaced from their original positions by a variety of causes, by frost action, by the acts of man, by decay, in fact, by a variety of accidents which should not be allowed to happen to property monuments.

Since 1920 a few members of the Nassau-Suffolk (N. Y.) Civil Engineers Association have been making a drive to secure effective standardization of surveying tapes. Permission has been obtained from the County Boards of Nassau and Suffolk to establish a bench standard in the basement of each Court House where there is little temperature change. It is proposed to place, in the concrete floor, bronze plugs at 0, 25, 50, 75, and 100 ft., so that if there is an error in a tape its amount and the quarter of the tape in which it occurs can be determined. The speaker has found from experience that a bend or repair is the principal source of change in length.

This summer (1927), the speaker made a survey of about 20 miles of ocean and bay frontage in Southern New Jersey. The original survey, covering all or part of the tract, was made by magnetic meridian about 1741. The point of beginning was then a monument set at the intersection of the center line of an unmonumented avenue and an unopened street. The tract was bounded on three sides by tide-water. When the present survey was begun, the State of New Jersey made a request to the U. S. Coast and Geodetic Survey, through the U. S. Bureau of Commerce and Navigation, to have the old Government triangulation re-established. The triangulation of the U. S. Coast and Geodetic Survey in that vicinity had been established

* Southampton, N. Y.

about 1873, but all points nearer than eleven miles to the region covered by the speaker's survey had been destroyed or lost. During the relocation of the old triangulation, three stations were located on the property which the speaker was surveying, one of them being on the north boundary. The location of this station on the boundary does not, of course, affect the old legal record, but it does make a monumented line which will become a part of the new record. It is certainly a fact that if all monuments relating to that tract of about eleven miles of ocean frontage are lost in the future, any competent engineer can go over on the stable ground on the opposite side of the Bay and from triangulation stations in that region relocate any part of the boundary of this property with an error of not more than a few inches.

The sooner surveyors adopt the principle of running all property surveys on rectilinear co-ordinates based on the true meridian and the National datum as established by the U. S. Coast and Geodetic Survey, the better it will be for the public and for the engineer. This applies not only to the property surveys of cities and towns, but also to townships and small country places.

With reference to the adoption of rectilinear co-ordinates, it is a very simple matter to deduce them from the spheroidal co-ordinates of Government triangulation. The U. S. Coast and Geodetic Survey has a publication which describes how this may be done, or at a nominal cost that Bureau will make the computation for a city or State. It is probably not permitted to do it for individuals or corporations.

Surveys should be established from the true meridian, as established by the U. S. Coast and Geodetic Survey, or by direct observations on Polaris made by an engineer. The speaker never makes a magnetic survey. It is more expensive to use the true meridian in the beginning, but in the end it is much cheaper for the client because when the survey is finished any competent engineer can relocate it very easily, especially if it is tied into the nearest triangulation stations of the U. S. Coast and Geodetic Survey, or of other Government organizations executing triangulation of equal accuracy based on the North American datum.

In the fall of 1925 at Montauk, N. Y., the speaker had occasion to resurvey a tract of about 12 000 acres from which 108 different parcels had been sold at various times. The original survey and each later separate survey had been tied by azimuth and distance into the triangulation of the U. S. Coast and Geodetic Survey. These surveys were completed in about four months. The time and cost would have been doubled had it not been for the old triangulation connections, because most of the original property corners had been moved or destroyed.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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MAXIMUM FLOOD DISCHARGE IN SAN JOAQUIN VALLEY, CALIFORNIA

Discussion*

By Messrs. Robert Follansbee and Harold K. Fox.

Robert Follansbee,† M. Am. Soc. C. E. (by letter).‡—The author states that, up to a certain elevation, ranging from 4 000 to 6 000 ft., precipitation increases with height, but higher than that, it decreases slowly to the summit. That this statement concerning a zone of maximum precipitation below the summit is not applicable to the Rocky Mountain region is shown by precipitation records in Colorado, so situated as to be comparable.

The first set presented comprises stations located on a section through the Continental Divide in Northern Colorado, on both eastern and western slopes, with one station at the summit. These records are summarized in Table 6.

TABLE 6.—Precipitation Records of Northern Colorado.

Slope.	Station.	Elevation, in feet.	Mean precipitation, in inches.	Length of record, in years.
West	Fraser	8 560	19.12	8
East	Longs Peak (near)	8 600	20.99	9
West	Spruce Lodge	9 600	30.13	6
East	Silver Lake	10 500	37.02	10
Summit	Corona	11 600	45.99	6

The second set are located near a section extending from Colorado Springs at the base of the mountains to the summit of Pike's Peak (Table 7).

* This discussion (of the paper by Oren Reed, Jun. Am. Soc. C. E., published in November, 1927, *Proceedings*, but not presented at any meeting) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Dist. Engr., U. S. Geological Survey, Denver, Colo.

‡ Received by the Secretary, November 25, 1927.

TABLE 7.—PRECIPITATION RECORDS IN COLORADO.

Station.	Elevation, in feet.	Mean precipitation, in inches.	Length of record, in years.
Colorado Springs	6 098	14.37	33
Glen Eyre	6 500	16.15	18
Fremont Experiment Station	8 850	22.79	10
Lake Moraine	10 265	24.98	24
Pike's Peak	14 107	29.55	15

Even a casual comparison of the two sets will show that elevation is not the only factor in determining the amount of precipitation, since that on Pike's Peak at 14 107 ft. is less than the precipitation at 9 600 ft. on the Continental Divide. The writer has shown elsewhere* that the controlling factor appears to be the presence or absence of mountain masses to be westward.

HAROLD K. FOX,† M. AM. SOC. C. E. (by letter).‡—The water records of California streams do not cover long periods. The best and oldest records have been taken at points where the streams enter the valleys (usually above irrigation intakes), and were primarily established for purposes of irrigation. In investigating power resources, the several branches and tributaries in the high altitudes are the points of interest, and it is only recently that records have been obtained on such streams. A large part of these records are less than ten years old and, since the past ten years have been somewhat under normal in precipitation, it is necessary to combine these records with those of longer duration on the main stream and to compare one river with another.

The rapid development of power has necessitated exhaustive studies of water resources and flood flow. The writer has a close personal knowledge of the vast amount of work that Mr. Reed has done in preparing this paper and hopes that it will be as beneficial to others who may have occasion to use it as it has been to those who developed the Kings River Project of the San Joaquin Light and Power Corporation. On this project it was necessary to locate the first power house in a very narrow and deep canyon, and one of the problems was that of protection against floods.

The writer has consulted the author on the matter of flood determination and finds that his method has been changed in certain details. The primary data are now used to determine one relation curve, which is called the "geographical location" curve. The location of a drainage area in relation to the path of cyclonic storms is of primary importance in this section of California.

The relation of average elevations and size of drainage area are, however, very important. Referring to Fig. 3,§ it will be noted that the elevation of maximum precipitation from the normal storm is about 6 000 ft. for Kings River. Storms which produce the highest run-off on the Kings River occur in January and February. At this season precipitation on the upper part of the water-shed is in the form of snow and contributes only a small run-off.

* *Water Supply Paper 500–C*, U. S. Geological Survey, p. 69.

† Supt. of Constr., San Joaquin Light & Power Corporation, Fresno, Calif.

‡ Received by the Secretary, January 3, 1928.

§ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2225.

Above Elevation 7000 the mean monthly temperature is below 30° Fahr., in January and February. During some days of these months the temperature would be above freezing, but the contribution to the run-off would be slight. Thus, the effective area would be much reduced. The effective area of the Kings River above Sanger would be about 900 sq. miles instead of 1 740 sq. miles.

The mean elevation of the effective area is then determined. The various streams will then be compared with respect to the relative location of the mean elevation of their effective areas. If the mean elevation coincides with the point of maximum precipitation, it will have a factor of 1.0. On the other hand, if it falls above or below the elevation of peak precipitation, the factor will be suitably lowered. In this manner, factors will be obtained for evaluating the effect of average elevation.

The observation that a stream of small tributary area will have a higher run-off per square mile than a large stream from a storm of given intensity has been the basis of many flood formulas. The effective area at the time of maximum flood expectancy should again be used. In order to eliminate the effect of difference in precipitation, only four streams will be used in determining this curve—the Kings, Kaweah, San Joaquin, and Merced Rivers.

In February, 1927, a storm occurred on the Kings River which produced a run-off of high intensity. The locus of the storm was over a relatively small area about five miles west of the junction of the North Fork with the main stream, or about eight miles from Balch Camp. The area had been burned over by a forest fire during the previous July. Periods of moderate rainfall preceded the main storm, which was of cloudburst intensity. Each small watercourse became a torrent and, since there was no grass or underbrush to hold the soil, erosion was very severe. Each watercourse became a gully, and boulders (many 10 ft. in diameter) were washed down, filling the channel of Kings River and forming pools of slack water. These dams sustained the flow very well and are only now being leveled down.

The gauge at Piedra showed a peak discharge from this storm of about 22 000 cu. ft. per sec. This discharge from the upper reaches of the stream was not high; the discharge of the North Fork at Balch Camp was about 2 000 cu. ft. per sec. A conservative estimate of the run-off from the main storm area is 10 000 cu. ft. per sec. from an area of approximately 100 sq. miles. This storm area was in the foothills, where there are moderate slopes.

It is not improbable that at some future date the locus of a similar storm will be over the North Fork area, where the run-off would be even more rapid, due to the steeper slopes and less cover. If that should happen, a run-off of 25 000 sec-ft. (or 101.5 cu. ft. per sec. per sq. mile) is entirely possible. A storm of this intensity will probably never occur before December, when the upper water-shed would contribute little to the flow. The run-off from the effective area might then be 150 cu. ft. per sec. per sq. mile.

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THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE

Discussion*

By Messrs. George Paaswell, C. C. Williams, and Herbert Chatley.

George Paaswell,† M. Am. Soc. C. E. (by letter).‡—In the writer's opinion, the author has done more for rational foundation practice than any other man or committee in the entire history of earthwork practice. An unusual combination of keen scientific and mathematical training, with an abundance of practical experience as a consulting and contracting engineer, has enabled Professor Terzaghi to make contact with actual problems at every stage of his theoretical investigations.

The paper, being synoptical in character, the author, naturally, omitted many vital points which bear on foundation practice as well as the type of laboratory and field tests which supplied the data for the theory developed. The paper must be a great disappointment to engineers who look for a simple set of rule-of-thumb notes to embrace all foundation practice. It is no mean accomplishment of the author that he has demonstrated the fact that foundation design and construction is as exacting in analysis and as worthy of study as the design of the structures resting on these foundations.

The essential difference between superstructure and sub-surface design is that one may fit the materials to the design of the former; one must fit the design to the materials of the latter. The engineer cannot fit a foundation to a rigid set of formulas. The material must be studied in the site and the foundation must be fitted exactly to it. A code set of bearing values is more than worthless; it is dangerous. The author has illustrated this point very well. Settlement in a foundation has been viewed as an unfortunate and usually unavoidable concomitant of foundation construction; yet such

* Discussion on the paper by Charles Terzaghi, M. Am. Soc. C. E., continued from January, 1928, *Proceedings*.

† Engr., Corson Constr. Corporation, Brooklyn, N. Y.

‡ Received by the Secretary, November 15, 1927.

settlement is as inevitable as the deflections and distortions of the supported structure. Provision must be made for them. That makes it necessary to formulate means to determine foundation distortions. The author has briefly indicated how such studies may be made.

The study of soils has indicated that two broad soil types exist: (1) Those in which the usual stresses of elastic solids are found, modified, of course by proper elastic coefficients; and (2), those in which hydro-mechanical stresses are set up. Both types can be uniquely identified by sample tests for composition and loading, but neither can be safely described by grain size or moisture contents alone. The author's striking experiment on sand and mica mixtures has demonstrated fully the futility of mere sieve analysis as a basis of soil classification. He has put to rest also the colloid theory as the basis of coherent action in soils.

In his paper entitled "Transmission of Pressure Through Solids and Soils and the Related Engineering Phenomena",* the writer attempted an analysis of stress distribution through soils and found that certain characteristic failure surfaces existed, similar in type to those found in elastic solids. It was also demonstrated that a uniform distribution of pressure under a load was a fiction of design. Numerous experiments preceding and following the paper have amply demonstrated the truth of this statement.

The huge subway construction program of Greater New York is unquestionably the world's best laboratory for the study and illustration of all types of foundation problems and the writer hopes to see, in the course of the next few years, the crystallization of a definite soil mechanics and the complete abandonment of bearing value codification as a foundation formulation.

The author well points out that, given a complete soil identification, there is sufficient mathematical and physical science available to complete the practical investigation, of the foundation. It remains to codify soil identification.

The use of piles has been a blind foundation expedient, save where such piles were driven to absolute refusal. The anticipated capacity of a pile whether determined by formula or by actual field test has been of so indeterminate a character that most foundation failures may be laid to miscalculated pile bearing values. It has been frequently emphasized by the writer that no adequate pile theory could be formulated until a rational soil mechanics had been developed, for the author has well demonstrated the fact that there is no separate theory for piles and for static soil theory. The development of the pile for underpinning buildings has evolved a better understanding of soil behavior than any other single phenomenon in engineering foundation practice. It is not a question of bearing capacity alone, but of actual load settlement behavior which determines the ultimate safety of the foundation underpinned. The study of the loaded soil and its distortions under the pile toe brought out the reasons for progressive settlement even if a pile had already been tested for a safe load and there was evolved the ingenious "pre-test" pile, a scientific application of soil mechanics.

The writer is heartily in favor of the abandonment of the *Engineering News* formula for piles, and the adoption of one similar in nature to that

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1563.

suggested by the author, although he would prefer to disregard pile formulas entirely and make a separate study for each foundation type. That would eliminate the dangerous conditions resulting from group application to single pile tests or formulas.

There remains the extremely interesting subject of temporary earthwork supports, such as sheeting and bracing. In this work modern soil mechanics has been given ample demonstration of its validity. The failure of earth banks is not a simple application of the Rankine or Coulomb special soil-type theory, but is one that can be easily formulated by a study of the elastic properties of the soil. The absolute limits of ground movement are predictable, and that gives a rational method for determining the maximum pressures that may be induced in the sheeting and timbering. This, of course, leads to a far safer and more economical layout of such bracing; but, on the other hand, it requires more study and more design than is usually given to such structures. The use of large excavating machinery, which requires large clearances because of working room, calls for a very careful investigation of load distribution through wales and braces. It must be borne in mind that a movement in the sheeting that permits movement of the earth behind it, also permits the grain re-arrangement of the soil and consequent increase in pressures as is well illustrated by the famous "dilatancy" experiment of Osborne-Reynolds.

C. C. WILLIAMS,* M. AM. SOC. C. E. (by letter).†—Professor Terzaghi has brought to the profession a valuable paper on an important phase of the science of foundations, and the principles of soil mechanics which he sets forth should serve as a rational basis for correlating observed facts concerning foundation tests and behavior, and should thus contribute to making the design of foundations more scientific.

That the supporting capacity of soil is not, in general, directly proportional to the area of the foundation, has long been recognized, but the true relationship is not even yet entirely understood. In experiments on alluvial soil made in 1851-52 at New Orleans, La.,‡ in which areas varying from 0.25 sq. in. to 576 sq. in. were used, the penetration under a constant unit load increased practically with the area. Observations made in France§ on alluvial soil indicated similar results. From these and other data,‖ the late Randell Hunt, M. Am. Soc. C. E., drew the conclusion: "Large areas of soft soil will not support as much weight per unit of surface as more limited areas of the same soil." Reflection on the manner of distribution of a local load through an indefinitely extensive solid or semi-solid, would indicate such to be the case.

The relation between the plastic and the elastic deformations of soil should first be recalled. The former is due to displacement of soil particles whereas the latter is due to the resilient deformation of the particles, without slipping, relative to each other. Contrary to the behavior of steel, the plastic flow pre-

* Dean, Coll. of Applied Science, Univ. of Iowa, Iowa City, Iowa.
† Received by the Secretary, December 9, 1927.
‡ *Van Nostrand's Magazine*, Vol. 27.
§ *Annales des Ponts et Chaussées*, 1864.
‖ *Proceedings*, Assoc. Eng. Soc., 1888.

cedes the elastic, the latter constituting a small proportion of the total compression. Fig. 17, taken from a test made under the writer's direction by Mr. H. T. Heald, a graduate student, illustrates this relationship for loam and for sand. The experiment consisted of applying and releasing a load which was increased with each application and never entirely removed, the compression and the recovery being recorded at each application.

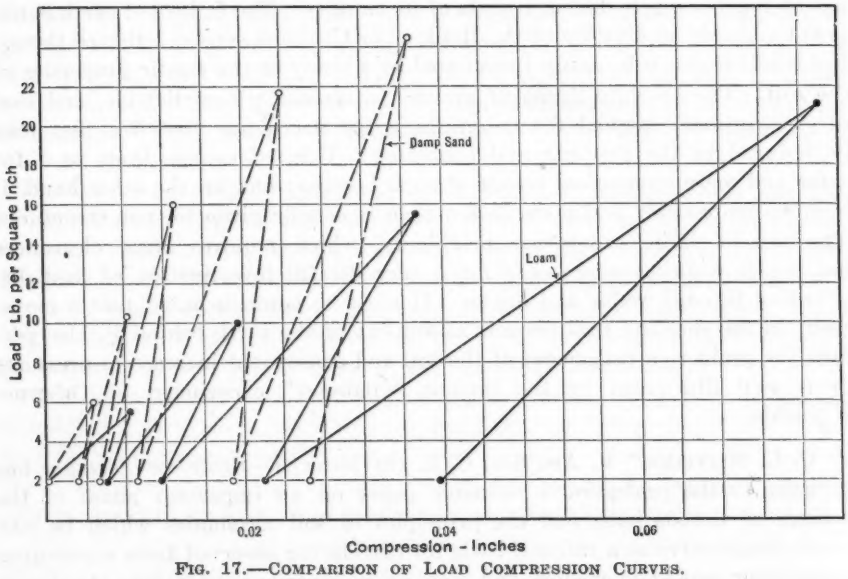

FIG. 17.—COMPARISON OF LOAD COMPRESSION CURVES.

If a load is applied over an area, A, at the surface, the inter-reactions of soil particles along the perimeter of this area are such as to spread the load laterally by virtue of shear over an ever-increasing area of bearing planes at successive depths. The rate of this spread depends on the characteristics of the soil, being greatest for coarse fragmental materials, less for granular, and least for silt. Moreover, the spread is only approximately linear and the surface of uniform pressure is curved rather than plane, but the variation from the conception indicated in Fig. 18 is not sufficient to affect seriously the conclusions based thereon. Although the rate of spread is dependent on the soil characteristics, the depth of soil affected, and hence the total spread, depends on the depression of the soil under the load. This deformation increases until the spread distributes the load over an area sufficient to bring the intensity within the elastic resistance of the soil. When a homogeneous soil is assumed, an expression for the total deformation may be obtained as follows:

Let W be the load applied; h, the total depth of soil compressed; y, the depth to any plane considered; r, the tangent of the angle of spread; and C, the modulus of compressibility. The distribution of pressure on any horizontal plane may be assumed equivalent to a uniform intensity over a width of $b + r\,y$. Then, for a square surface with side, b, the deformation of an elementary horizontal prism bounded by the sides of the frustrum of the

pyramid is $\frac{W\,d\,y}{C\,(b + r\,y)^2}$, and the total deformation in a depth, h, is $\frac{W}{C}$ $\int_0^h \frac{d\,y}{(b + r\,y)^2}$, which when integrated gives $\frac{W\,h}{C\,(b^2 + b\,r\,h)}$. This expression shows that the deformation varies inversely with the perimeter, (b), as well as with the area, (b^2).

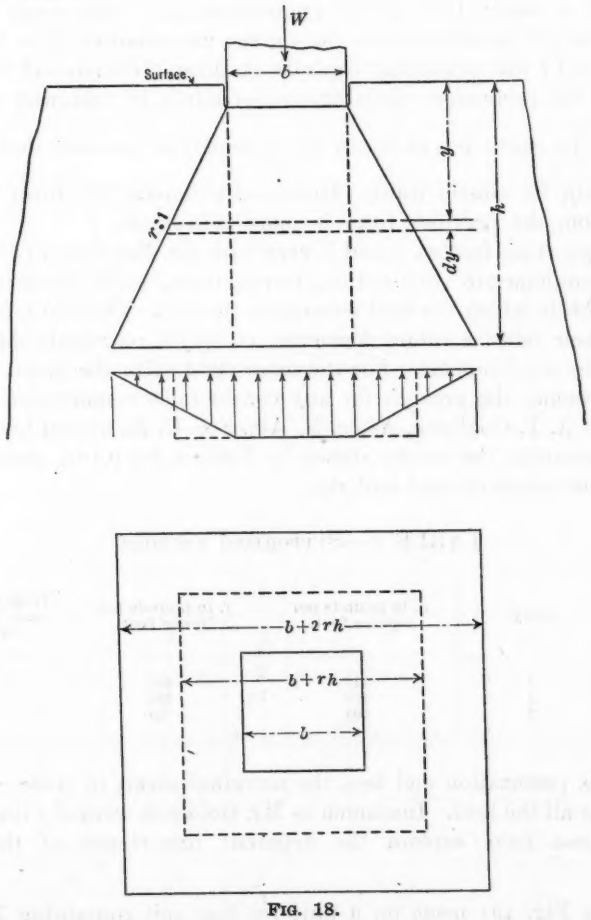

FIG. 18.

If w is the pressure intensity at the surface and p the intensity that the soil will sustain elastically without deformation (at a given depression), then, $w\,b^2 = p\,(b + r\,h)^2$, whence $h = \frac{b}{r}\left(\sqrt{\frac{w}{p}} - 1\right)$, neglecting the weight of the soil itself. If, as for sand under certain conditions, $r = \frac{1}{2}$, and $h = 2$, the deformation would be $\frac{2\,W}{C\,(b^2 + b)}$. In this case the denominator contains simply the sum of the bearing area and the perimeter multiplied by a strip $\frac{1}{4}$ ft. wide.

Thus, an elementary analysis indicates that the supporting capacity of soil depends on two factors: (1) The bearing area; and (2) the perimeter of the bearing surface. However, on account of the variableness of soils, it is impossible to assign values that will be universally applicable, to these factors. The relationship may be stated as a formula for convenience,

$$W = A\,p + L\,f$$

in which, W = load; A = area of application; p = supporting capacity per square foot of soil in compression for a given deformation; L = length of the perimeter; and f the supporting capacity in shear (friction and cohesion) per foot around the perimeter. This expression might be otherwise conceived as $\left(A + \frac{L\,h\,r}{2}\right)p$, since p is in reality the intensity of pressure that the soil will sustain within its elastic limit. However, although this form may clarify the conception, the previous form is more convenient.

These supporting factors, p and f, vary with the character of the soil (mineral and granulometric composition, compactness, water content, etc.), and with the depth to which the load penetrates the soil. The two terms are indeterminate, their relative values depending on which represents the more rigid element in the total support. For the more rigid soils, the direct compressive resistance becomes the greater, for any considerable compression. Thus, the tests made by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., quoted by the author,* give, approximately, the results shown in Table 2 for 0.1-in. penetration and for the various mixes of sand and clay.

TABLE 2.—SUPPORTING FACTORS.

Sand.	Clay.	p, in pounds per square foot.	f, in pounds per linear foot.	Total superimposed load, in pounds per square foot.
0	1	110	400	1 710
1	1	120	280	1 240
5	1	500	160	1 140

At 0.05-in. penetration and less, the marginal shear, in these experiments, carried nearly all the load. Inasmuch as Mr. Goldbeck worked with small compressions, these facts explain the apparent importance of the diameter $(\sqrt{A})$.

Tests (see Fig. 19) made on a uniform clay soil containing 7% sand in undisturbed condition, using bearing areas of 1.0, 2.25, and 4.0 sq. ft., showed for 0.3-in. depression, p = 3 500 and f = 1 500. For depressions of less than 0.1 in., the perimeter factor carried nearly all the load. In still another test in clay soil, p = 3 400 and f = 2 300 for a settlement of $\frac{3}{8}$ in. While the Austrian experiments quoted by the author† are rather fragmentary, the corresponding values at 1.0-in. settlement are approximately, p = 2 800, and f = 700.

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2266.

† *Loc. cit.*, p. 2265, *et seq.*

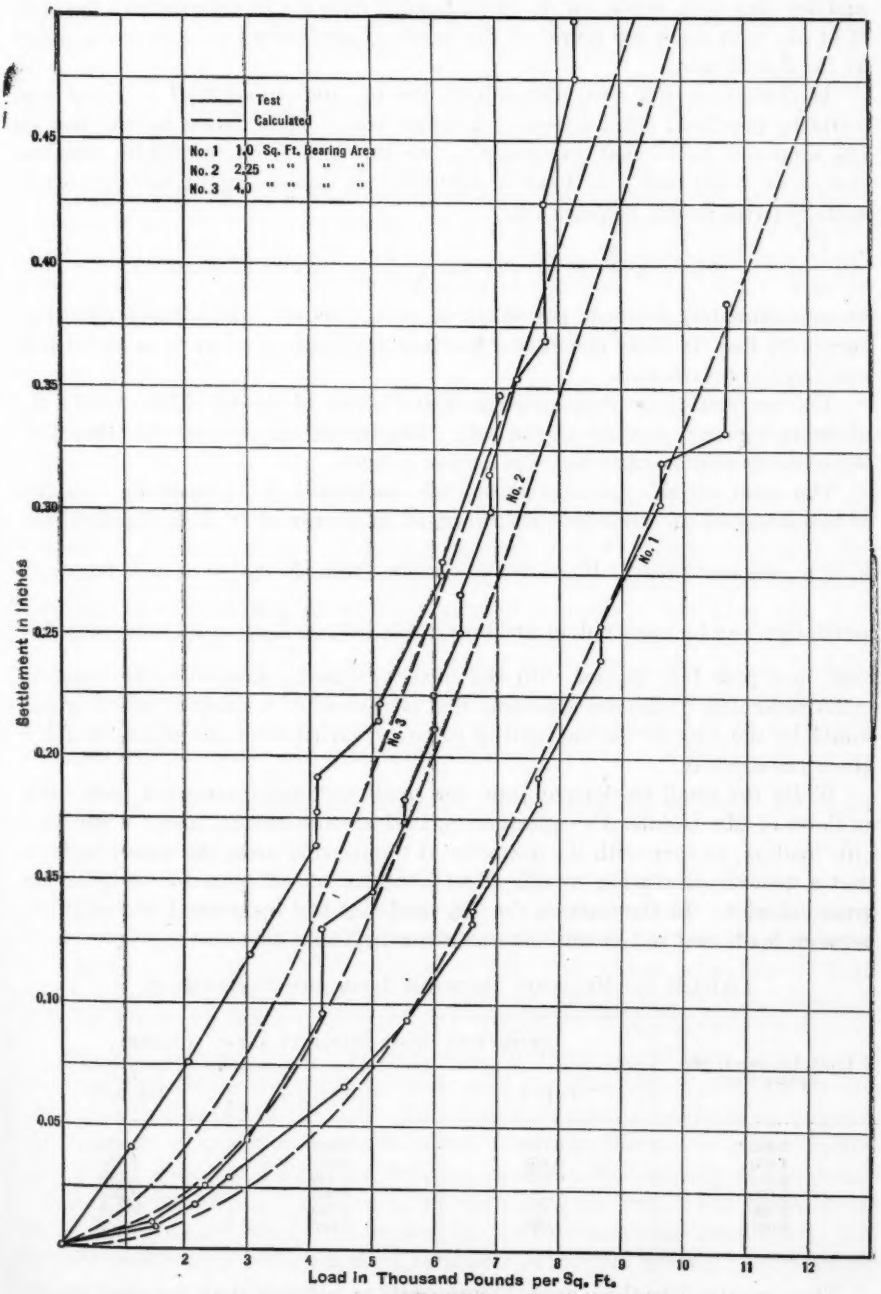
Test
Calculated
No. 1 1.0 Sq. Ft. Bearing Area
No. 2 2.25 " " " "
No. 3 4.0 " " " "
No. 1
No. 2
No. 3
Settlement in Inches
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
2
3
4
5
6
7
8
9
10
11
12
Load in Thousand Pounds per Sq. Ft.

FIG. 19.

Of these two elements, p varies almost directly with the settlement in sand and dry clay soils, while, on the other hand, f remains fairly constant, varying, if at all, with some low power of the depth of settlement, in some cases about as the fourth root.

If these elements were determined, the bearing capacity of a larger area could be predicted from a test on a small area. Thus, based on the test on 7% sand soil mentioned previously, using the data from the 0.3-in. penetration, $p = 3\,600$ and $f = 1\,500$, a footing 8 ft. square might be expected to sustain (with 0.5-in. penetration),

$$3\,600 \times \frac{5}{3} \times 64 + 1\,500 \sqrt[4]{\frac{5}{3}} \times 32 = 432\,000 \text{ lb.}$$

or, approximately 5 400 lb. per sq. ft. of superimposed load. The broken-line curves of Fig. 19 show calculated bearings assuming p to vary as the depth, and f as the fourth root.

The correlation between calculated and observed data is high within the ultimate bearing capacity of the soil. The writer has applied this theory to the other test data available, with similar results.

The most reliable practical basis for comparing the supporting capacity of two areas on the same soil, therefore, is the factor, $A + K \times L$, in which, K is a soil coefficient. If, as for a certain sand, K is $\frac{1}{4}$, then a 36-sq. ft. foundation can be expected to sustain on this soil, $\frac{36 + 6}{1 + 1}$, or 21 times the test load on a post 1 ft. square with the same settlement, instead of 36 times as much according to the usual assumption, or instead of 6 times as much, which would be the case if the supporting capacity varied with the diameter for a given penetration.

While for small settlements and for small test areas, observed data, such as those of Mr. Goldbeck's experiments, indicate settlement, under a constant unit loading, to vary with the diameter of the bearing area, the writer believes that a general conclusion to this effect is unwarranted, even for soils having great cohesion. In the tests on the 7% sand-clay soil mentioned, the relations between loads and settlement were as given in Table 3.

TABLE 3.—Relation Between Load and Settlement.

Load, in pounds per square foot.	Settlement Under Different Areas, in Inches.		
	1 sq. ft.	2.25 sq. ft.	4.00 sq. ft.
3 000	0.032	0.042	0.112
4 000	0.054	0.082	0.155
5 000	0.075	0.132	0.210
6 000	0.110	0.205	0.265
7 000	0.152	0.325	0.340
8 000	0.225	0.385	.....

These results, like those quoted by the author, indicate that, for small settlement, and small areas, the movement is approximately proportional to the

diameter, but that, for larger loadings and larger areas, this relationship tends to disappear. This latter indication is further supported by the fact that, in the test on clay, the settlement under actual load of 5 500 lb. per sq. ft. on 68 sq. ft. was only about 1.5 times that under the same unit load on the test post covering 4 sq. ft., instead of 4 times as great as would have been the case if the settlement had been in proportion to the diameter.

In the seventeen-story office building of the Michigan Central Railway Company at Detroit, Mich., settlement under a bearing post, 2 ft. square (not back-filled) was 0.02 ft. for 5 000 lb. per sq. ft., but under the mat foundation of the building loaded to 4 000 lb. per sq. ft. there was no appreciable settlement.* Indeed, numerous instances might be cited where an actual load on large footings amounts to one-half or even a larger fraction of a test load on a small area, and yet the settlement is not noticeably greater than that with the test load.

Where care is exercised to load the soil around the test footing, the discrepancy between the settlement under test and actual load on the structure is reduced. Unfortunately, too few correlated observations have been made on the settlement of the completed structure, and that of the test load at similar intensities of pressure. It is the writer's opinion, however, based on the rather meager data available, both theoretical and observational, that, for areas of practical size, the settlement does not vary greatly for comparable-sized footings carrying a uniform intensity of pressure on the same soil.

If, as seems to be the case from this discussion, the bearing capacity varies with both the area and the perimeter of the footing, then, for larger foundations, the bearing for constant settlement varies nearly with the area. For instance, with two footings on sand, as previously assumed, one 10 ft. square and one 20 ft. square, the respective bearing capacities would be about as $\frac{100 + 10}{400 + 20} = 1 : 3.8$, instead of 1 : 4 when based on area directly, or 1 : 2 if based on the diameter relationship.

While both theoretical analyses and experimental observations show, as the author states,† that the bearing intensity is not uniform under a pier, it is the writer's opinion that the error in the moment calculated for a continuous beam or mat, on the assumption of uniform distribution, is not as serious as that shown in Fig. 3. The author neglects the fact that such a foundation beam or slab deflects upward, and that this deflection is generally sufficient to relieve the pressure at the mid-span, and, therefore, operates to effect more nearly a uniform distribution, by increasing the intensity of pressure at the (downward) supports of the slab. This deflection upward operates to decrease the pressure at mid-span, because of the fact that the soil is only slightly elastic and does not follow the deflected member, the loading of the beam differing therein from a gravity or constant-pressure load. For spans of 16 to 20 ft., or more, which the writer has had occasion to investigate, this upward deflection of the beam is sufficient to nullify greatly the error in moment on the assumption of uniform distribution.

* *Engineering News*, September 3, and October 1, 1914.

† *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2270.

Two other phases of the science of foundations, both suggested by the author,* enter into foundation design to an important degree. These are: First, the relative effect on settlement of a constant dead load and of a transitory conjectural live load to which foundations are subjected; and, second, the prediction of foundation behavior in order to calculate the effect of settlement on the superstructure. One has but to visit the basements of almost any of the older structures in Chicago, Ill., and cities similarly situated, to witness the effects of lack of knowledge on this subject. Eccentric live loads, due to wind and other elements, are usually unavoidable, and a prediction as to the behavior of the foundation under such conditions and the consequent effect on the superstructure is necessary in order to prevent distortion, if not failure, of the structure. To secure uniform minimum settlement under a structure may be desirable, just as rock foundations may be desirable; but, since such foundations are not always obtainable, the problem then resolves itself into designing the structure for actual foundation conditions. Therefore, studies of soil behavior should include more detailed information than the usual observations accompanying test loading.

Professor Terzaghi's stimulating paper should bring out in discussion a great deal of information concerning soil mechanics and its relation to the foundation design, as well as other phases of the science of foundations.

HERBERT CHATLEY,† ESQ. (by letter).‡—The writer has had the privilege of following Professor Terzaghi's work for several years. Whether or not some of the finer points of Professor Terzaghi's hypotheses will stand prolonged investigation is a matter for the future to decide, but no one can question that his masterly exposition of the manner in which the flow of water through fine-grained soils is related to their bearing capacity has thrown a brilliant light on what had previously been a most misunderstood subject.

The conditions in Shanghai, China, are those of a saturated soft alluvium of indefinite depth, and the major conclusions drawn by Professor Terzaghi as to the behavior of surface loads and piling under such conditions are of the greatest interest and value to engineers in Shanghai. Through the initiative of H. von Heidenstam, M. Am. Soc. C. E., pile tests on rather a large scale have been made for some years in Shanghai by the staff of the Whangpoo Conservancy Board. Professor Terzaghi has been good enough to speak very appreciatively of these tests. Broadly speaking they have confirmed his views as to piling in soft soils.

The slowness with which fine alluvium expels its contained water is well shown in Shanghai by the behavior of hydraulic fills formed by the pumping up of dredge spoil. Where these fills are above the ground-water level they do become dry, but below the top 2 or 3 ft. some years may elapse before the deeper material becomes fully consolidated. This is true in spite of the fact that the hydraulic filling process tends to remove an appreciable part of the colloid content.

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2263.

† Acting Engr.-in-Chf., Whangpoo Conservancy Board, Shanghai, China.

‡ Received by the Secretary, December 30, 1927.

As to the use of piling formulas, the cogency of the reasoning as to the variation of the hydro-dynamic resistance with the duration of the applied force is complete. The new "derivation" of the *Engineering News* formula is most interesting and calls attention to the fact that the assumed constancy of the second term in the denominator of that formula is even more arbitrary than is generally supposed.

In the work on the internal friction of granular masses the author has seen fit to adopt the writer's conclusion* as to the importance of single molecular bonds in uniting particles the weight of which is of the same order as those bonds. The writer has since gone much further into the question and these studies fully sustain his earlier conclusions as to the magnitude and importance of these forces in granular masses.

In regard to the classification of soils the new data as to the effect of mica are most important. There is one point to which attention might very well be directed and that is the possibilities of change of grain size by the permanent aggregation of particles. Putting it very crudely, one might imagine that, under certain circumstances, a fine clay could gradually be transformed into a loam by the adhesion of the particles.

Engineers would be very glad if the author would give some practical rules for the design of sheet-piling, which acts as a retaining wall.

* "On Cohesion", *Philosophical Magazine*, Vol. XL, August, 1920, and "Silt", *Minutes of Proceedings*, Inst. C. E., Vol. CCXII, p. 400.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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THE CONOWINGO HYDRO-ELECTRIC DEVELOPMENT ON THE SUSQUEHANNA RIVER

Discussion*

By H. A. Hageman, M. Am. Soc. C. E.

H. A. Hageman,† M. Am. Soc. C. E.—Mr. Wilson has given the general features of the hydraulic design of the Conowingo Project.‡ It is the purpose of this discussion to set forth in somewhat greater detail the design of the major parts of the structure and operating equipment.

Station Intake.—On account of the length of waterway required to accelerate the water and the width of the space necessary to house the electrical bay, the intake is in itself a stable structure. It will serve adequately as a dam and no support from the power station substructure is necessary. For the four future units, this makes necessary, at present, the construction of the intake only, and results in a minimum initial expenditure. It will also make possible the future realization of all progress in water-wheel design which may be made prior to the installation of these units.

Trash Racks.—As Mr. Wilson has explained, the entrances to the waterways of the turbines will be located near the bottom of the reservoir and a curtain-wall extending 40 ft. below the normal pond surface will be constructed up stream from them as a protection against ice. The curtain-wall is a distinct feature of the intake and is more economical than any other form of ice protection of equal efficiency. Each waterway will be divided into two parts from the entrance to a point a short distance up stream from the butterfly valve, where they will be merged into one.

The trash racks are located in each intake opening a sufficient distance behind the ice-protection wall to permit the operation of a mechanical rack rake. These racks will be made up in panels of convenient size, which will slide in guides set in the concrete. The lower panels in each opening will be

* Discussion on the paper by Alexander Wilson, 3d, Esq., continued from November, 1927, *Proceedings*.

† Chf. Hydr. Engr., Stone & Webster, Inc., Boston, Mass.

‡ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2295.

equipped with flat steel bars, but the upper panels, which are located behind the curtain-wall and are, therefore, of no use in providing actual waterway area, will be faced with treated timber. The timber facing on the panels will decrease the adherence of ice to the racks at the water surface and also reduce the cost of the panels.

The rack sections have been designed for an unbalanced head of 5 ft. over their entire area, under which conditions the maximum stress in the steel will not exceed 16 000 lb. per sq. in.

The guides for the emergency sectional head-gates will be located down stream from the trash racks. These gates will be of structural steel and will be used to close the waterways when it is desired to unwater them in order to inspect or repair the butterfly valves. The valves are used normally as head-gates.

A sufficient number of gate sections will be provided to close the intake of one unit. The top, or sealing section, will be arranged to seal against a casting at the top of the intake opening so that it will not be necessary to extend these gates to the water surface.

Steel stop-logs of similar design to the head-gates, but with sections of less height, will be provided to close the waterway openings for the future units until such a time as these units are installed.

Gantry Cranes.—The sectional head-gates and trash racks will be handled by the gantry cranes which travel along the intake and spillway sections of the dam. For this purpose the cranes will be equipped with a special lifting beam which will run in the guides for either the head-gates or trash racks. This lifting beam will attach automatically to a rack panel or head-gate section under water, so that each section may be hoisted out of the guides separately and transported to a suitable storage space by the cranes. When it is desired to place the head-gate, trash-rack, or stop-log sections in the intake the process will be reversed, the lifting beam automatically releasing the sections when they have reached their proper place.

The wide bellmouth provided for the waterways insures very satisfactory hydraulic conditions for the intake with the head loss at the entrance reduced to a minimum.

Velocity at Intake.—With the water-wheels discharging 5 300 cu. ft. per sec. corresponding to full generator load at 89-ft. head, the velocity through the trash racks will be about 2.6 ft. per sec. The velocity at the entrance of the waterways will be 2.8 ft. per sec. In the distance of 71 ft. from the entrance of the waterways to the center line of the butterfly valves the water will be accelerated to a velocity of about 12 ft. per sec. After passing the center line of the butterfly valve, the velocity decreases and the water enters the plate-steel scroll case of the water-wheels at about 9.3 ft. per sec.

Butterfly Valves.—The area of the entrance to the water-wheel intakes for each unit is about 1 900 sq. ft. To close this area with two or even three lift-gates of any developed type would require gates of enormous size for the pressure to be resisted, with the consequent difficulty of obtaining an adequate seal against leakage. The operating mechanism required would be expensive and the operation slow.

By placing a circular gate or butterfly valve at the entrance to the scroll case, the area to be closed is only about 570 sq. ft. as compared to the 1 900 sq. ft. for rectangular gates. Further, the perimeter to be made water-tight is only about 85 ft. for the circular gate compared to 350 ft. for the rectangular gate.

Butterfly valves, 27 ft. in diameter, of the vertical shaft, circular type, were selected as being the most practicable to suit the conditions. They will be designed to open or close in about 5 min. and may be closed with maximum flow through the turbine. In the closed position they will stand at right angles to the line of flow. Each valve will be sealed after closing by admitting water pressure to a rubber tube, 3 in. in diameter, set in the inside face of the valve housing opposite the periphery of the valve wicket. Based on tests of this sealing device, the leakage through these enormous valves will be negligible. Each valve will be operated by oil pressure in a cylinder 40 in. in diameter.

These valves will be the largest ever built, the largest now in existence being the valves, 23 ft. 6 in. in diameter, built by the Allis-Chalmers Manufacturing Company for the Niagara Falls Power Company. For the Conowingo development, three valves are being built by The Wm. Cramp and Sons Ship and Engine Building Company and four by the Allis-Chalmers Manufacturing Company, each manufacturer supplying the valves to serve as the head-gate for the water-wheels he is to furnish. The sealing device will be the same for all valves and the manufacturers have co-operated in the design of bearings, operating mechanism, and renewable parts, making them alike and interchangeable.

Water-Wheels.—The Allis-Chalmers Company and The Wm. Cramp Company are co-operating in the design of the water-wheels, with the result that only one set of principal spare parts will be required. The outward appearance of the units will also be similar. To this end, such parts as lignum vitæ guide bearings are of the same design for all wheels and are interchangeable. The runners, while not of the same design, will have the same limiting dimensions, so that the spare runner which will be supplied by The Wm. Cramp and Sons Ship and Engine Building Company, will serve in either a Cramp or Allis-Chalmers setting with high efficiency.

It is expected that the main water-wheel units will have a maximum efficiency of at least 93% and also that, in an average year of run-off, the seven main generators will have an energy output of 1 382 000 000 kw-hr. Plate-steel scroll cases were chosen on account of their greater economy when compared to concrete which would require heavy reinforcing in the substructure and much more elaborate and expensive form work. The maximum diameter of the runner will be 17 ft. 9 in., and its weight, when completely assembled, will be about 240 000 lb. Each main shaft will be of forged steel about 36 in. in diameter, with top and bottom flanges forged integrally with it.

A structural steel pit liner and generator support will be provided for each main unit. It will be attached to the speed ring of the water-wheel and be designed to carry the weight of the generator, thrust-bearing load, and adjacent floor loads down to the speed ring, whence the load is transferred through the speed-ring vanes to the power station substructure. The support

will be built of plate and structural steel instead of cast iron, as is ordinarily the case. The support is essential to the power station design as it was found that it would be very difficult to design a concrete foundation for the generator in the space available to properly transfer the enormous load to the speed ring through which it must be transmitted to the substructure foundation.

Governors.—Each water-wheel manufacturer will supply with the unit which he furnishes, actuator type, oil-pressure, relay governors of his own make, having governor flyballs mechanically driven from the main shaft of the water-wheel.

Power House.—The units and auxiliaries will be arranged in pairs. Units Nos. 1, 2, 5, and 6 will have Allis-Chalmers turbines and General Electric generators. Units Nos. 3, 4, and 7 will have Cramp turbines and Westinghouse generators.

Each pair of units will be equipped with an interconnected oil-pressure system consisting of one pressure tank and duplicate pumps and sump tanks. The governor actuators and pressure tanks will be located on the main floor of the station at Elevation 46.0 ft., but the pumps and sump tanks will be located on the station auxiliary floor at Elevation 35.0 ft.

A central oil-purification system will be provided with the necessary piping to drain any governor system and purify the oil and return it to the system.

Draft-Tubes.—The draft-tubes for each make of water-wheel will be built in accordance with the design of their respective manufacturer. The Cramp units will have tubes of the conventional Moody type with a concrete cone extending from the bottom at Elevation — 14.5 ft. to the hub of the runner. The Allis-Chalmers units will be equipped with a White Hydraucone type of draft-tube and will have its bottom at Elevation — 18.5 ft. The bottom of the outlet of the draft-tube will be at the same elevation for all units.

The substructure over the draft-tube of each unit will be supported by five cast-steel stay vanes which each water-wheel manufacturer is furnishing for the units he supplies. Down stream from the circle of stay vanes, the draft-tube will be divided into two parts by a central bearing wall.

The low setting of the water-wheels relative to the tail-water level, required for the protection of the runners against pitting, makes it necessary to provide for stop-logs at the outlet of the draft-tubes to prevent the unit from being flooded by tail-water when unwatered for inspection or repair. These stop-logs will be handled by a small gantry crane in a manner similar to the sectional head-gates and trash racks.

All waterways will be arranged with suitable connections to a drainage tunnel which will be located under the inspection tunnel, up stream from the draft-tubes. This drain will empty into a sump at the west end of the station from which the water will be pumped into the tail-race.

Spillway.—As already explained by Mr. Wilson,* the head-water level at the station is to be maintained constantly at Elevation 108.5 ft. except during periods of extreme drought, at which time the draft from the reservoir will cause the head-water to fall below the normal elevation. In order to pass

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2298.

flood flows in excess of the requirements of the water-wheels a spillway equipped with crest gates will be provided.

The spillway is of the conventional ogee type, having a parabolic crest and a circular curved surface at the toe. The piers supporting the crest gates are 7 ft. thick and 45 ft. on centers, leaving a clear opening of 38 ft. between them. These piers also support the highway and the runway for the gantry cranes that operate the gates.

Crest Gates.—The crest gates will be of the Stoney type, 22 ft. 6 in. high by 40 ft. wide, with timber seals at the bottom and rubber-covered staunching rod seals at the sides. Fifty crest gates will be provided, having a combined discharge capacity of about 800 000 cu. ft. per sec., with the head-water level at 108.5 ft. Three smaller regulating gates, each having a capacity of about one-third that of a crest gate, will be provided for the finer adjustments of flow. Up stream from the crest gates, additional guides will be set in the piers to receive a service gate. This gate will be of the fixed roller type and will be used to close the gate-opening when it is necessary to remove one of the crest gates or regulating gates for painting or repairs.

The crest gates, regulating gates, and service gates will all be handled by the same cranes that handle the sectional head-gates. A total of three gantry cranes will be provided for handling gates on the dam and the sectional head-gates and trash racks. For handling the crest gates they are equipped with special lifting beams of different design from that used to handle the sectional head-gates and stop-logs. As this lifting beam is never submerged, it is arranged for an operator to ride on it and operate the locking device which attaches it to the gates. When the gates are raised to their open position they will be held in place by means of pawls engaging suitable stops on the ends of the gates, thus permitting the crane to move on to the next gate.

A study of the duty cycle of these cranes indicates that one crane will lift a crest gate every 9 min. As the maximum rate of rise of any flood on record has been such as to require the opening of one gate only every 30 min., ample factor of safety has been provided in these gantry cranes to insure the timely opening of these gates.

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EMERGENCY DAM ON INNER NAVIGATION CANAL AT NEW ORLEANS, LOUISIANA

Discussion*

BY MESSRS. F. R. HARRIS AND R. H. WHITEHEAD.

F. R. HARRIS,† M. AM. SOC. C. E.—This is a very interesting contribution to the very little that has been written on this subject. Such structures are not only necessary and an insurance against possible very serious damage to locks, amounting in some cases even to their entire destruction, but they are an adjunct necessary to enable periodic unwatering and inspection, together with the necessary repair and maintenance work involved.

The stop-log emergency dam and canal lock is new to the speaker. Mr. Goldmark states that at an earlier period an emergency dam was built on the Canadian side of the St. Mary's River, and that it was actually tested under emergency conditions. Figs. 1‡ and 2‡ of the accident are very interesting.

However, the feature of making steel girder stop-logs capable of being floated to any location is a novel one. The speaker is not sure that he entirely understands how water is admitted to them to overcome their buoyancy when lowered into position. That is, whether this is an automatic device or a special hand preparation. It would seem that when lifting them from the water, unless the openings are very large to permit the interior water to be discharged, great care would have to be exercised so as not to strain the lifting truss or the hoisting mechanism, unless, of course, the designer had provided for this by additional strength for the additional weight of water.

The speaker would anticipate some difficulty in securing and maintaining water-tightness of the box girders. He does not know whether these have been provided with caulking edges and caulked, or whether the water-tightness

* This discussion (of the paper by Henry Goldmark, M. Am. Soc. C. E., published in December, 1927, *Proceedings*, and presented at the meeting of December, 7, 1927), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Rear-Admiral, U. S. N. (*Retired*); Cons. Engr., New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2621.

is dependent on gaskets. In either case, when these stop-logs have been subjected to load and consequent deflection, it would seem that the joints would start leaking. It would be interesting to know what experience had revealed in this line.

The author's hooking and unlatching device is very ingenious and would require careful attention so that corrosion may not occur and interfere with its operation. This apparatus, which is principally for emergency use, may lie unused for long periods, and the temptation is to neglect proper inspection and upkeep. When the emergency arises the apparatus is often not in proper condition. There is no way to overcome this except to insist on periodic use and placing of the apparatus for inspection purposes, whether it is or is not actually required. Unless something of this kind is done, it is either not on hand or not in usable condition when required.

The speaker has used similar stop-log dams, although of much smaller dimensions, in the discharge culverts of dry docks. Their purpose in this case is a somewhat similar one. They are provided for use in the event of the breakage of a sluice-gate or gate-valve, in order to prevent reverse flow of the water and back-race of pump impellers, together with the incident flooding when flooding is not desired. Also, they are used as an emergency dam to permit inspection and repair of discharge culverts and chambers. As they are much smaller, with less head and span, they are much simpler and can be improvised. The speaker has had several experiences of deciding to use them and then, after specifying them, making the discovery that they could no longer be found. The Dock Master had gotten tired of seeing them hanging around so long and not being used and had sent them to the scrap heap or had broken them up and used the material for other purposes.

It would be interesting to know something about the extent of leakage in horizontal joints between abutting girders or stop-logs. The leakage in this type of emergency dam cannot, presumably, be greater than that at Panama. On the other hand, when considering this type of dam as compared to the type of emergency dam at Panama, attention must be given to the greater span of the Panama locks and the consequent greater depth of girder (horizontally), greater bending moment, and consequent greater weight.

Considering the theoretical design of an emergency dam a series of horizontal beams, such as Mr. Goldmark has designed, will have to be designed to take the entire water pressure. In general, the steel required and the cost would bear a direct relation to the width of the opening and the depth (horizontally) of the beams. In the case of the Panama emergency lock, there is one horizontal girder at the top having a span the full width of the opening. The sum of the reactions received by it from the vertical members, or beams, will equal one-third the water pressure, the remaining two-thirds being taken by the sill. The vertical beams, in turn, since the combined maximum moment, would be a distance of one-third up, in a general way—all other factors being equal—the steel structure would have to be designed to take care of two-thirds of the water pressure on the basis of the Panama emergency dam as compared to 100% for a stop-log dam. It is, of course, very probable that the details in the Panama design as compared to the relative simplicity

in the stop-log design might more than make up this difference, although in comparing costs with the New Orleans costs—attempting to equalize the difference in cost on account of the conditions existing at the different times of construction—the greater depth and span, in the case of Panama, should generally result in higher cost.

It is difficult to compare the costs of a design completed before the World War, with one made under existing conditions, because the cost of labor and materials is so much higher.

Going back again to the question of water-tightness for flotation purposes, it has occurred to the speaker that in the long span of 75 ft. there would necessarily be some deflection due to water pressure. What effect would this have on water-tightness, especially after continued repetition of deflections? The matter of water-tightness is of considerable importance in considering the stability of these box girders because they are at times a floating structure, and it is evident that when so floating, the center of gravity is above the center of buoyancy, giving a type of stability dependent entirely on the righting effect, which is computable in terms of moment of inertia of the water plane of flotation. If the girders should leak to any extent or contain any water the effect would then be that of interior water careens with consequent instability and a tendency to roll over on the side. If this does occur, and it will occur with leakage, there will be some difficulty in attaching the lifting hooks. The speaker would like to inquire if anything like this occurred.

The paper was also not very clear as to whether the process of flooding the gates and making them water-tight was automatic or whether it was a hand operation. At one time it is stated that they are arranged so as to freely take water in order that they can be lowered and at another time that they must be water-tight.

As an alternate suggestion to prevent leakage in horizontal joints, the speaker recommends making the meeting surfaces of a soft wood, such as white pine. The Federal Government has been very successful in using that material instead of rubber gaskets for meeting surfaces of gates.

R. H. WHITEHEAD,* ESQ. (by letter).†—The Panama Canal type of dam is an emergency dam, pure and simple. The stop-log type described by Mr. Goldmark has the merit of not only being ready as an emergency dam, but the girders used take the place of the very expensive floating caisson of the Panama Canal. If the latter type of emergency dam had been used in the Inner Navigation Canal at New Orleans, it would have been necessary to supplement it with a floating caisson for overhauling the lock-gates at the upper and lower ends of the locks, assuming that the operating policy was to guard the equipment against deterioration. From the standpoint of simplicity, there is no doubt that the stop-log type is simpler than the Panama type; but the writer does not regard this of very great importance, because it is the business of the operating force in charge to master any details of operation and to be ready to respond with efficiency to any emergency. How-

* Vice-Pres. and Gen. Mgr., The New Haven Clock Co., New Haven, Conn.
† Received by the Secretary, January 7, 1928.

ever, when it comes to maintenance, the Panama Canal type requires more careful maintenance to keep it in a high state of efficiency, and this is expensive and demands close supervision.

The choice of types of dams, either the stop-log or the Panama Canal type, depends on the size of the opening to be closed. The writer does not believe that the stop-log type, while it seems to be a very good type for New Orleans, would be the correct type to use at Panama. The importance of the Panama route makes expense secondary and of minor importance, and speed of operation in an emergency of major importance.

Mr. Goldmark states that it requires 1 hour to put the Panama Canal in position.* On the contrary, from the time the emergency occurs it requires, at a maximum, only ½ hour. It takes an hour to put the smaller New Orleans Dam in position, and it is evident by considering the designs that, basically, the Panama Canal type is a faster one to operate. It is true that it was originally expected that an hour would be required to put the Panama Canal Dam in position, but the operating force, by the proper indexing of the clutches required for handling the gates in storage, and by proper training, has reduced this time by half.

The Panama Canal type of dam is safer for Panama than the stop-log type, because the area is blocked off by separate independent units in a vertical plane rather than separate dependent units in a horizontal plane.

In regard to Mr. Goldmark's method of calculating the cost on a square-foot basis and comparing the two types, it would probably be a better comparison if the two dams were calculated on a tonnage basis, because after all is said and done, if the New Orleans type of dam, which is 75 ft. wide, was built to cover a width of 110 ft., such as in Panama, the cost per square foot would mount up much more rapidly than in direct proportion to the relative areas. In calculating on a pound basis, it would be found on certain sizes of openings that the Panama Canal dam, considered simply as a dam, is cheaper when the area becomes sufficiently large, than the stop-log type, and *vice versa*. As the area becomes smaller from a critical amount, the stop-log type will show more and more economy. There appears little doubt but that the stop-log type is the correct one for New Orleans and that the Panama Canal type is the best for locks of their size.

The writer wishes to pay tribute to the author. The operating force of the Panama Canal has always looked on the lock-gates that Mr. Goldmark designed and that the McClintic-Marshall Company built as the greatest single engineering achievement of the Panama Canal.

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2640.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

REDMOND STEPHEN COLNON, M. Am. Soc. C. E.*

DIED OCTOBER 9, 1927.

Redmond Stephen Colnon, the son of John W. and Harriet (Perkins) Colnon, was born at Brownville, N. Y., on June 29, 1862. After his preliminary education, Mr. Colnon entered Cornell University, at Ithaca, N. Y., from which he was graduated in 1887, with the degree of Civil Engineer.

Soon after his graduation from college, he was engaged as Assistant Engineer in the office of the late Emil Kuichling, M. Am. Soc. C. E., at Rochester, N. Y. In 1889, he became Assistant Engineer with Charles H. Ledlie, M. Am. Soc. C. E., of St. Louis, Mo., and, in 1890, occupied a similar position with the firm of Johnson and Flad, in charge of the construction of an inclined cable, at an estimated cost of $75 000, for the Western Railway Company. From 1892 to 1895, he was employed as Engineer with the Fruin-Bambrick Construction Company, General Contractors, in charge of an inshore tunnel on the Water-Works Plant for St. Louis. From 1895 to 1900 he was engaged in various construction work.

In 1900, Mr. Colnon entered into partnership with Mr. Jeremiah Fruin under the firm name of Fruin and Colnon, General Contractors. With this corporation, his work included the construction of a warehouse for the St. Louis Transfer Company and a water intake plant for the Anheuser-Busch Brewing Association, as well as city paving and sewers for St. Louis and East St. Louis, and the building of new settling basins for the City Water-Works at Chain of Rocks, Mo.

In 1908, the Fruin-Colnon Contracting Company was organized. This Company specialized in public works, railroad contracting, etc., and, from 1912 until his death, Mr. Colnon served as its President.

He was a member of the St. Louis School Board from 1903 to 1909. He was appointed a member of the St. Louis Terminals Commission in 1905, and was a member of the St. Louis Library Board from 1926 to his death. He was also a Director and Vice-President of the Missouri Portland Cement Company, and a Director of the Merchants Laclede National Bank, of the Scullin Steel Company, and of the St. Louis Frog and Switch Company.

Mr. Colnon was a member of the following social and fraternal organizations: The Noonday Club, the Racquet Club, the Circle Club, the Engineers' Club of St. Louis, the City Club, the Bellerive Country Club, the Sunset Hill Country Club, Theta Delta Chi Fraternity, Cornell University Alumni Association, and the St. Louis Council of the Knights of Columbus.

He was married on January 9, 1895, to Katherine Fruin, of St. Louis, who survives him.

Mr. Colnon was elected an Associate Member of the American Society of Civil Engineers on July 9, 1906, and a Member on July 12, 1926.

* Memoir prepared from information on file at Society Headquarters.

HUBERT EDWARD GREEN, M. Am. Soc. C. E.*

DIED SEPTEMBER 7, 1927.

Hubert Edward Green was born at Edgbaston, Warwickshire, England, on August 17, 1859. He received his grammar school and college education in Auckland, New Zealand.

From 1874 to 1884, Mr. Green was with the Public Works Department of the New Zealand Government, on the construction of its railway system. In June, 1880, he was appointed Overseer, in charge of wharf building, and of road and bridge repairs, and in July, 1882, he was made Office and Field Assistant.

In 1884, Mr. Green came to the United States, and from February of that year until September, 1886, he served as Construction Engineer for the Sierra and Mohawk Valley Railway Company, a narrow-gauge line running west into California from a point about 30 miles north of Reno, Nev. During the following two years, he was in charge of the City Engineer's office at San Diego, Calif.

In 1888, Mr. Green entered the employ of the International Company of Mexico, with headquarters at Ensenada, Lower California, and made explorations to classify land over the Company's concession of 18 000 000 acres. He was engaged as head of the Railroad Engineering Corps and was employed on a railway survey from San Quintin to Yuma, Ariz., branching at Real del Castillo to San Diego. Considerable difficulty was experienced due to scarcity of water and transportation over the heavy sands through the Cocopo Desert and the intense heat.

Returning to the United States in 1890, he had charge of the construction of the railroad trestle across Elliott Bay, for the Seattle Terminal Railway Company. In February, 1891, he served on a location and canal survey for the Lands Department of the Northern Pacific Railway Company, between North Yakima and Prosser, Wash. He was also engaged from 1891 to 1893 on the location of railroad lines in Texas for the Atchison, Topeka and Santa Fé Railroad Company from Conroe, Tex., to the Trinity River. In July, 1893, he was placed in charge of levels, with the Imperial Valley and Colorado River Irrigation Company, under C. R. Rockwood, Chief Engineer.

From 1893 to 1896 Mr. Green was with the San Diego City and County and Coronado Beach Company, superintending the laying of the foundation of the Morena Dam, which extended 100 ft. below the stream bed. During 1897 and 1898 he was engaged in placer mining in British Columbia and in the general practice of civil engineering in San Francisco, Calif. In 1899, he was employed on the construction of electric railroads in Los Angeles, Calif., and from 1900 to 1902, he acted as Field Assistant for the United States Geological Survey.

During 1902 and 1903 Mr. Green served as Assistant to J. B. Lippincott, M. Am. Soc. C. E., in charge of surveys, estimates, and designs for pipe lines

* Memoir prepared by M. M. O'Shaughnessy, M. Am. Soc. C. E.

for the Seaside Water Company, of Long Beach, Calif. While engaged in this capacity, he made divisional surveys of the Hetch Hetchy Valley and Lake Eleanor Reservoir sites for the City of San Francisco.

On May 4, 1903, he was appointed Designing Engineer of the United States Reclamation Service, in charge of the construction of the Belle Forche Diversion Dam. He held this position until his resignation in 1907. From 1907 to 1910, he designed and built irrigation works for La Sierra Heights, at Riverside, Calif.

During 1910 and 1911 he was employed by the Mountain Power Company, of Crescent City, Calif., in making surveys and designs for the power dam on the Smith River, and from 1912 until his death, he acted as Chief Engineer for the Compañia Constructora Richardson, at Esperanza, Sonora, Mexico, where he died on September 7, 1927.

Mr. Green was a man of sterling character and ability, modest and retiring, the reason, perhaps, why he did not occupy as conspicuous a position as his abilities warranted. He will be greatly missed by his many engineering friends in California.

It was the writer's pleasure, as a young man, to work under Mr. Green on the construction of the Sierra Valley and Mohawk Railway, in Plumas County, California, in 1885. He then learned to have great respect for his keen knowledge as an engineer. Since that time, during a period of forty years, he has had many friendly experiences with him. He was possessed of great patience, but was firm in his decisions, as was illustrated by the difficulties he overcame in laying the foundation of the core of the Morena Dam, when he was representing the City of San Diego. At this time he encountered opposition from ignorant foremen who wished to use inferior concrete, which course Mr. Green, with his careful engineering training, resisted.

Mr. Green was twice married, first, to a young woman from San Francisco, who died in 1907, and, subsequently, to a New Zealand friend of his youth, who survives him.

Mr. Green was elected a Member of the American Society of Civil Engineers on December 2, 1903.

OSCAR HOLMES TRIPP, M. Am. Soc. C. E.*

DIED AUGUST 24, 1927.

Oscar Holmes Tripp was born January 15, 1852, at Waterboro, Me., the son of the Reverend Leander and Louisa A. Tripp. His father was a Baptist clergyman and his early years are difficult to trace because of the migrations made necessary by this calling. His youth was spent in Hancock County, Maine, where he received his primary education in the Public Schools. He afterward attended one of the Maine academies and later studied law, being admitted to the Hancock County Bar, and subsequently practicing at Bluehill, Me. As was not uncommon in the earlier days, he carried on the practice

* Memoir prepared by Walter C. Groves, Assoc. M. Am. Soc. C. E.

of surveying in conjunction with the practice of law, but gradually a keener interest in the problems of engineering drew him from the legal profession and it was as an engineer that he was known.

From 1881 to 1884, Mr. Tripp was employed as Assistant Engineer on the Bangor and Katahdin Iron Works Railway, and the Bangor and Piscataquis Railroad. Both these roads are now part of the Bangor and Aroostook System. In 1885, he opened an office at Rockland, Me., for the general practice of engineering, and except for occasional short engagements in other sections of Maine, and fifteen months of the years 1898 and 1899 spent as Assistant Engineer in the Sewer Department at Arlington, Mass., he was located at Rockland. During this period, he served for several terms as City Engineer.

He was an assiduous reader, with a taste for literature and history as well as science. He was very observant and this, coupled with the fact that he made a record of his observations and the happenings of the day, gradually made his office a treasure house of accurate information on local history.

Mr. Tripp was a member of the Knox County Bar Association and, for several years, had served it most efficiently as Treasurer and Librarian. He was a member of the Congregational Church. He was a Scottish Rite and a York Rite Mason. He was also a member of the Boston Society of Civil Engineers.

He died suddenly of heart failure in the Houston-Tuttle Book Store in Rockland, while browsing, as was so often his custom, among the new books. For several years Mr. Tripp had been in ill health and none realized as well as he, how near death hovered. Nevertheless, he continued his work up to the day preceding his death. He was unmarried, and his nearest surviving relative is a niece, Mrs. Flora T. Cooper, of Portland, Ore.

Mr. Tripp was elected an Associate Member of the American Society of Civil Engineers on October 7, 1896, and a Member on September 5, 1905.

JOHN MICHAEL LALLY, Affiliate, Am. Soc. C. E.*

DIED OCTOBER 8, 1925.

John Michael Lally was born in Lockport, N. Y., on July 22, 1865. His parents, Michael and Elizabeth Lally, later moved to Detroit, Mich., where his early youth was spent.

Mr. Lally's first professional work was with his father's construction company, known as the McRae, Lally and Sons Company. He was associated with this company in various capacities ranging from Instrumentman to Superintendent of Construction. From 1881 to 1893, he was in charge of construction for the Michigan Central, the Pere Marquette, and other railroad companies, as well as water-works, docks, and dredging on various projects, in Michigan, Wisconsin, Pennsylvania, and New York.

From 1893 to 1894, he was Chief of Party in charge of a preliminary survey and estimates for a railroad from San Salvador to La Libertad in Central

* Memoir prepared by L. C. Hammond, M. Am. Soc. C. E.

America, and from 1894 to 1898, he had charge of the construction of a dam at Sault Ste. Marie, Mich., for the Michigan Central Railroad Company, a 10-mile extension of the Chicago Stockyards, and track operation improvements in the vicinity of Detroit. During this same period, he placed 25 miles of railroad lines for the Hocking Valley Railway Company in Jackson, Mich., and an extension for the Pittsburgh, Shawmut and Northern Railroad Company, in Southern New York; also, 11 miles of double track on the New York Central Railroad, near Depew, N. Y.

From 1899 to 1906, Mr. Lally was engaged as Superintendent of Construction and Supervisor of Track on the New York Central Railroad, at Buffalo, N. Y., where he had charge of the territory from Batavia, N. Y., to Tonawanda, N. Y., including the Buffalo Railroad Yards. In 1906, he became General Superintendent in charge of construction of 140 miles of railroad for the J. G. White Company, General Contractors, from Raleigh to Washington and Newberne, N. C., which position he retained until 1909.

From 1909 until his death, he was in charge of construction and engineering work for the New York Central Railroad Company, and the following year he was Superintendent in Charge of Improvements on the Electric Zone. During the period of Federal Control, he was attached to the Corporate Organization of the System's various lines, inspecting and reporting on their condition. After the World War, and previous to his death, he was Assistant Engineer on the staff of the late George Alec Harwood, M. Am. Soc. C. E., at that time Vice-President in Charge of Improvements and Development of the New York Central Lines.

Mr. Lally was a man of sterling character, and exceptional ability as a Construction Engineer. All his life he had charge of large construction work with an organization of loyal men. His ability for keeping the good will and confidence of the men who worked for him, helped him to secure excellent results in his work. Many younger men who received their experience under his direction remember him with gratitude and respect for the patient and kindly help which he was always ready to give them.

Death came suddenly from heart disease at his home, in Yonkers, N. Y. The funeral services were held in St. Denis Church, Yonkers, with the Reverend Joseph Cameron, of Rochester, N. Y., the Reverend Ignatius Cameron, of Geneva, N. Y., brothers-in-law of the deceased, and the Reverend James Doris, of Yonkers, officiating. Burial was in Detroit. He is survived by his wife, Mary C. Lally, and by three brothers and two sisters living in Detroit.

Mr. Lally was elected an Affiliate of the American Society of Civil Engineers on September 2, 1908.

HOWARD AUGUSTUS SHERIDAN, Jun. Am. Soc. C. E.*

DIED AUGUST 2, 1927.

Howard Augustus Sheridan was born in Elizabethport, N. J., on December 17, 1900. In 1906, his parents moved to Toronto, Ont., Canada, where he

* Memoir prepared by A. Radtke, Jr., Esq., Detroit, Mich.

attended the Clinton Street School, until 1910, when his family changed their residence to Buffalo, N. Y. He was graduated from the Public Schools, and entered Lafayette High School at the age of twelve years. In his Junior year, his father moved to a fruit farm in New York State, and the boy was transferred to the Lockport, N. Y., High School, from which he was graduated at the age of sixteen years. He helped on the farm until the following summer, when he attempted to enlist at Buffalo, but was rejected because of his age.

Mr. Sheridan then enrolled in the Reserve Officers Training Corps, at Ann Arbor, Mich., and on its disbandment, returned to Buffalo, where he took a business course at the Bryant and Stratten College. In 1920, he re-entered the University of Michigan, which he left the following year in order that he might teach school at Niagara Falls, N. Y., but he returned again to Ann Arbor in 1923. During his attendance at the University, he was prominent in campus activities, having been a member of Web and Flange (Honorary Civil Engineering Society), Tau Beta Pi, and the Student Chapter of the Society, of which he was President. He was graduated in 1925, with honors and a degree in Civil Engineering.

Immediately after his graduation, Mr. Sheridan entered the employ of the F. R. Patterson Construction Company, of Detroit, Mich. A few months later, he transferred to the Gannett, Seelye, and Fleming Company, at Harrisburg, Pa., where he remained until the spring of 1926, when he returned to the F. R. Patterson Construction Company to serve as Engineer on the Fordson High School Building. From December, 1926, until the time of his death, he was Assistant General Superintendent of the Patterson Company.

He was killed on August 2, 1927, while on his way to work, when his car was struck by a train at Dearborn, Mich.

Mr. Sheridan proved himself a capable and efficient engineer, and while serving as Assistant General Superintendent for the F. R. Patterson Construction Company he showed considerable ability as a construction organizer. He was unmarried.

Mr. Sheridan was elected a Junior of the American Society of Civil Engineers on October 12, 1925.